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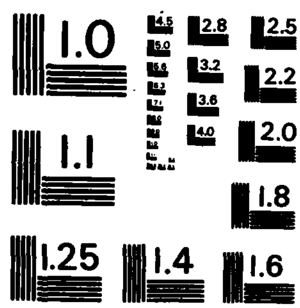
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Investigation of the FAA Overlay Design Procedures for Rigid Pavements

AD-A135 317

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Final Report

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16. Abstract

The existing FAA overlay design procedures and their history of development are first briefly presented in the report. This is followed by a detailed summary of consultant's reports, which include the identification of deficiencies in the existing procedures with suggested improvements. Immediate improvements to several items in the existing procedures are presented for use as new paragraphs and modification of existing paragraphs in the FAA Advisory Circular. Items to be addressed in a Phase II continuation study are presented.

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METRIC CONVERSION FACTORS

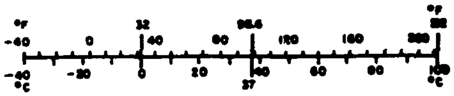
Approximate Conversions to Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
in	inches	*2.5	centimeters	cm
ft	feet	30	centimeters	cm
yd	yards	0.9	meters	m
mi	miles	1.6	kilometers	km
		AREA		
in^2	square inches	6.5	square centimeters	cm^2
ft^2	square feet	0.09	square meters	m^2
yd^2	square yards	0.8	square meters	m^2
mi^2	square miles	2.6	square kilometers	km^2
	acres	0.4	hectares	ha
		MASS (weight)		
oz	ounces	28	grams	g
lb	pounds	0.45	kilograms	kg
	short tons (2000 lb)	0.9	tonnes	t
		VOLUME		
tsp	teaspoons	5	milliliters	ml
Tbsp	tablespoons	15	milliliters	ml
fl oz	fluid ounces	30	milliliters	ml
c	cups	0.24	liters	l
pt	pints	0.47	liters	l
qt	quarts	0.95	liters	l
gal	gallons	3.8	liters	l
ft^3	cubic feet	0.03	cubic meters	m^3
yd^3	cubic yards	0.76	cubic meters	m^3
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C

* 1 in. = 2.54 (exactly). For other exact conversions and more detailed tables, see NBS Misc. Publ. 286, Units of Weights and Measures. Price $2.25, SD Catalog No. C13.10:286.

Approximate Conversions to Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
mm	millimeters	0.04	inches	in
cm	centimeters	0.4	inches	in
m	meters	3.3	feet	ft
m	meters	1.1	yards	yd
km	kilometers	0.6	miles	mi
		AREA		
cm^2	square centimeters	0.16	square inches	in^2
m^2	square meters	1.2	square yards	yd^2
km^2	square kilometers	0.4	square miles	mi^2
ha	hectares (10,000 m^2)	2.5	acres	
		MASS (weight)		
g	grams	0.035	ounces	oz
kg	kilograms	2.2	pounds	lb
t	tonnes (1000 kg)	1.1	short tons	
		VOLUME		
ml	milliliters	0.03	fluid ounces	fl oz
l	liters	2.1	pints	pt
l	liters	1.06	quarts	qt
l	liters	0.26	gallons	gal
m^3	cubic meters	35	cubic feet	ft^3
m^3	cubic meters	1.3	cubic yards	yd^3
		TEMPERATURE (exact)		
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F



PREFACE

The report presented herein was sponsored by the Federal Aviation Administration (FAA), U. S. Department of Transportation, through Inter-Agency Agreement No. DTFA01-81-Y-10523, "Update Overlay Thickness Criteria."

The study was conducted by the U. S. Army Engineer Waterways Experiment Station (WES), Geotechnical Laboratory (GL). Dr. Yu T. Chou wrote this report under the general supervision of Dr. W. F. Marcuson III, Chief, GL, Dr. T. D. White, Chief, Pavement Systems Division (PSD), GL, and Mr. H. H. Ulery, Jr., PSD. Mr. R. S. Rollings, PSD, prepared the materials in Appendix A. The FAA Project monitor was Dr. Aston McLaughlin.

The technical contributions of the following individuals are gratefully acknowledged: Professor Carl L. Monismith of the University of California at Berkeley, Professor Michael I. Darter of the University of Illinois, Professor Robert L. Lytton of Texas A&M University, and Professor Walter P. Kilareski of the Pennsylvania State University.

Commanders and Directors of WES during this investigation and preparation and publication of this report were COL Nelson P. Conover, CE, and COL Tilford C. Creel, CE. Technical Director was Mr. F. R. Brown.

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INTRODUCTION

BACKGROUND

The pavement overlay design procedures now used by the Corps of Engineers and the Federal Aviation Administration (FAA) were developed from accelerated traffic tests conducted during World War II and the following years. These procedures are empirical and have not been reevaluated in depth since approximately 1960. The FAA, realizing the importance of pavement overlays in upgrading and rehabilitating existing airport pavements, entered into an Inter-Agency Agreement (IAA) with the U. S. Army Engineer Waterways Experiment Station (WES) to review existing overlay design methods to determine if improvements could be made. To carry out the work WES formed a team consisting of personnel of the WES Geotechnical Laboratory's Pavement Systems Division and four other independent investigators. The latter group is comprised of the following members:

a. Professor Carl L. Monismith, The University of California at Berkeley.

b. Professor Michael I. Darter, The University of Illinois.

c. Professor Robert L. Lytton, Texas A&M University.

d. Professor Walter P. Kilareski, Pennsylvania State University.

Overlay test data accumulated by the U. S. Army Corps of Engineers was first compiled by Rollings[1-4] at WES and copies were sent to the investigators. Each investigator separately studied the data and prepared a report.[5-9] Copies of each report were made and sent to other members for review. A short summary prepared by Rollings of the Corps of Engineers' overlay test data including references is presented in Appendix A.

The team convened at WES, Vicksburg, Miss., on 9-10 February 1982 for the purpose of discussing overlays for rigid pavements and making recommendations to the FAA. The objectives of the investigation were to identify shortcomings in the present FAA overlay design procedure, to determine methods of improving overlay design, and to recommend research required to develop and validate an airport overlay design procedure that would yield equivalent performance for flexible and rigid overlays on a rigid base pavement.

During the Conference, each of the five team members presented a summary of the highlights of his investigation and recommendations and then answered questions and led discussions. At the completion of the Conference, the team

prepared a combined report[10] to the FAA based on the recommendations and the consensus of the discussions. The report is presented in Appendix B of this report.

PURPOSE

The purpose of this report is to summarize the reports prepared by the members of the Board of Investigators[59] and to provide recommendations for entry into FAA Advisory Circular[11] and a research plan for a future continuation of the study.

SCOPE

The existing FAA overlay design procedures and their history of development are first briefly presented, followed by a detailed summary of the reports by the team members, which includes the identification of deficiencies in the existing procedures and suggested improvements. Immediate improvements to several items in the existing procedures were presented which results in suggested new paragraphs and modification of existing paragraphs in the FAA Advisory Circular. Items to be addressed in a Phase II continuation study are presented.

EXISTING FAA OVERLAY DESIGN PROCEDURES

The current FAA overlay design procedure is contained in the Advisory Circular, AC 150/5320-6C, "Airport Pavement Design and Evaluation," December 7, 1978.[11] The circular is intended to provide guidance on the structural design and evaluation of airport pavements. It includes the design of rigid pavements and flexible pavements. Bituminous concrete and portland cement concrete (PCC) pavements placed on an existing pavement are considered by the FAA for overlay, but in accordance with the specific requirements of the FAA for the work reported herein only two types of overlays are discussed. These are bituminous concrete overlay and PCC unreinforced jointed overlay placed over an existing unreinforced jointed PCC slab. Regardless of the type of overlay, the following information concerning the existing rigid pavement must be obtained to design the overlay:

- a. Foundation conditions - soil classification, drainage, foundation support (subgrade modulus or k-value).
- b. Thickness of each layer, its condition, and its strength (flexural strength of concrete).
- c. Condition of the existing concrete slab (distress survey).
- d. Future aircraft traffic volume, type, and weight.
- e. Other factors - joint design and spacing, sealant condition, localized failures, etc.

This information is then utilized in overlay thickness design equations. The overlay equations and the history of their development are presented in the following paragraphs.

BITUMINOUS OVERLAY DESIGN

FAA places several restrictions and requirements on the use of bituminous overlays. A granular separation course between the old and new surface is not permitted as the layer may become saturated with water and provide unpredictable performance. The minimum thickness for a structural improvement is 3 in. Overlay thicknesses greater than the concrete base slab thickness should be designed considering the overlay as a flexible pavement and treating the existing rigid pavement as a high-quality base material. Reflection cracking in bituminous overlay is recognized as a source of problems; however, no recommendations are provided for prevention or reduction of reflection cracking.[11]

A new design thickness of PCC using the flexural strength of the existing pavement is calculated using current design methods. This thickness is then modified (decreased) by a factor F based on the amount of "allowable cracking" in the existing slab at the end of the service life of the overlay. In other words, the design method assumes that "a controlled degree of cracking" will take place in the existing slab.[12] The effective thickness of the existing slab can also be modified (decreased) using the condition factor C_b to reflect existing deterioration. The structural design equation for bituminous overlays is:

$$t = 2.5(Fh_d - C_b h_e) \tag{1}$$

where

- t = thickness of bituminous overlay, in.
- F = factor that controls the degree of cracking that will occur in the base pavement; it is a function of traffic and subgrade strength. Essentially, the F factor selected will dictate the final condition of the overlay and the rigid base pavement. Values range from 0.6 to 1.0.
- h_d = single slab thickness required for new design to be placed on the existing foundation.
- C_b = condition factor of the existing concrete slab. It is 1.0 when existing slabs contain nominal initial cracking and 0.75 when the slabs contain multiple cracking.
- h_e = thickness of existing concrete slab, in.
- 2.5 constant = equivalency factor relating concrete overlay thickness to bituminous overlay thickness.

The development of Equation 1 is described in Reference 13. The initial equation, which did not include the coefficient C_b , was developed in the mid-1950's based on accelerated traffic tests on six test tracks. Fifty-three test items were included in the six test tracks. The concrete base thickness varied from 6 to 12 in., and the bituminous overlay thickness varied from 3 to 42 in. (some of the overlays were more than five times the thickness of the base slabs). A range of subgrade strengths was also included in the test program. Twenty-six of the test sections were full-depth asphaltic concrete (AC); the remainder included plant mix black base with an AC surface, water-bound macadam with an AC surface, stabilized crushed rock with an AC surface, and sand asphalt base with an AC surface.

In the tests, the empirical data from the performance observations included determinations of the permanent deformation and general condition of the overlay surface and concrete base pavement at three stages during the traffic life. These were (a) prior to failure of the overlay; (b) at incipient failure of the overlay; (c) following the complete breakup of the overlay surface. Failure was determined by visual observation and corresponded to the first signs of visible deflection. It was found from the numerical data that this visible deflection of the surface corresponded to a point of rapid increase in measured deflections of the concrete base pavement.[13]

During the tests it was observed that the slab under the bituminous overlay started cracking soon after traffic was applied and continued until the average size of slab pieces reached 5 to 7 sq ft* without apparent effect on the overlay surface. If traffic were continued beyond that point, the deflections would increase drastically and shear failures would occur in the foundation. The basic premise behind the design equation was to determine thickness of bituminous overlay which would insure that the base pavement would not be broken into pieces smaller than the critical size during the design life of the overlay. The design equation also insures that a "complete failure" would occur in the base concrete pavement at the end of its service life; this is done by the use of F factor. Since more severe cracking can be tolerated on high-strength subgrades before shear failure occurs, and the rate at which cracking develops on high strength subgrades is slower (see Reference 12), the F factor is related to subgrade strength.

Information from other tests on performance of plain concrete pavements was used to define "complete failure" curves. These are shown in Figure 6 of Reference 13. The curves show the number of coverages to complete failure for percent of design thickness for various subgrade strengths. Although the test data were only for 5,000 coverages, the curves were extended to 30,000 coverages. These curves were used along with actual performance data to determine the concrete design deficiency. The bituminous overlay thickness used in the test was plotted versus the calculated concrete design deficiency (shown in

* The U. S. Army Corps of Engineers defines three levels of slab failure: (a) initial failure when a slab contains a single crack; (b) shattered slab when cracking divides the slab into six pieces; and (c) complete failure when cracking divides the slab into individual pieces having area of less than about 15 to 20 sq ft each.

Figure 11 of Reference 13). A design line was placed on this figure which has a 2.5-to-1 ratio of bituminous overlay to calculated concrete deficiency.

In Reference 14 the equation for thickness design of bituminous overlays was presented as:

$$t = 2.5(Fh_d - Ch) \tag{2}$$

C is described as a condition factor with the following numerical values: (a) $C = 1.00$ when the rigid base pavement slabs contain only nominal initial cracking, and (b) $C = 0.75$ when the rigid base pavement slabs contain multiple cracks and numerous corner breaks. No mention was made in the paper of where the C factor came from or how it was developed. The following assumptions were made during the development of the formula:

a. The accelerated traffic tests provided accurate information for use in design.

b. The gear configurations, gear loads, and operational characteristics used in the tests provided sufficient information for use in design.

c. There is no difference in performance between full-depth overlays and those constructed which incorporated high-quality base materials.

The design equation has remained unchanged and is in the current FAA Advisory Circular.[11]

CONCRETE OVERLAY DESIGN

The design of a rigid overlay on an existing rigid pavement is also based upon a rigid pavement design as determined by the procedure for a new pavement. Three types of concrete overlays are as follows:

a. Bonded, which requires careful surface preparation to ensure that full bond is achieved.

b. Partially bonded, in which the concrete overlay is placed directly on the existing concrete with little surface preparation other than minor cleaning.

c. Unbonded, which uses a leveling course of AC between the concrete slabs to prevent bond.

The minimum allowable thickness of a bonded concrete overlay is 3 in. The minimum allowable thickness of partially bonded or unbonded concrete overlays is 5 in. Specific jointing conditions are also specified. The determination of a new single slab thickness for the existing foundation requires the measurement of a modulus of subgrade support (or k-value) from a field plate-bearing test or estimation based on construction records. The flexural

strength of the existing concrete slab must also be determined by removing and testing specimens from the slab. The structural integrity of the pavement must be assessed and a condition factor selected. The condition factor is a coefficient which is used to reduce the effective thickness of the existing slab.

The structural design equation for concrete overlays of rigid pavements is:

$$h_o^n = h_d^n - C_r h_e^n \tag{3}$$

where

n = 1.0 tor fully bonded overlays

n = 1.4 for partially bonded overlays

n = 2.0 for unbonded overlays

h_o = required thickness of concrete overlay, in.

h_d = required single slab thickness above existing foundation determined from FAA design curves,[11] in.

h_e = thickness of existing concrete slab, in.

C_r = 1.0 for existing pavement in good condition--some minor cracking evident but no structural defects

C_r = 0.75 for existing pavement containing initial corner cracks due to loading but no progressive cracking or joint faulting

C_r = 0.35 for existing pavement in poor structural condition--badly cracked or crushed and faulted joints

Restrictions on the use of the different overlays include using bonded concrete overlays only on existing pavements which are in good condition with a C_r of 1.0. Partially bonded concrete overlays should be used only on existing pavements with a C_r of 0.75 or better. Unbonded concrete overlays are normally used on existing pavements with a C_r between 0.75 and 0.35. The leveling course in unbonded concrete overlay must be a highly stable bituminous concrete.

The development of the concrete overlay equation is presented in the following paragraphs.

UNBONDED OVERLAYS

The required thickness of an unbonded overlay currently used by the Corps of Engineers[15] is determined by the following formula:

$$h_o^2 = h_d^2 - C_r h_e^2 \quad (4)$$

The original source showing the use of the factor $n = 2.0$ cannot be traced. The following statements are directly quoted from the American Concrete Institute publication[16] concerning the applicability of Equation 4 for unbonded overlays:

> Until recently there has been no comprehensive analysis of stresses involved in multiple layers of concrete pavements similar to the Westergaard and Pickett studies of stresses in a single slab. It is not known by whom† or when the suggestion was first made for use of the formula which assumed that the structural capacity of two slabs, one superimposed on the other, is equivalent to that of a single slab the square of whose thickness is equal to the sum of the squares of the two slabs. . . . This formula came into use with the full understanding that it was not technically accurate. It may be approximately correct under the conditions that (1) the two slabs have the same stiffness, and (2) that there is no friction between them. Since it is most unlikely that the two slabs will be of the same stiffness, it is to be expected that one will be stressed more than the other. This is offset by the fact that, normally, considerable friction will exist between the two slabs which will cause them to act to some degree as an integral unit and thus reduce the stresses below what they would be if the two acted separately with no friction between them.

The footnote used in Reference 16 is as follows:

> †In a paper "Highway Research in Illinois," published in Transactions, ASCE, V. 87, 1924, p. 1209, Fig. 17, Clifford Older, chief highway engineer for Illinois, used a similar formula for evaluating monolithic brick sections of the Bates Test Road. It must be noted that these were not concrete slabs resurfaced with brick, but were built in such a way that there was a substantial bond between the concrete and the brick layers. In other words, an attempt was made to produce a monolith, hence the term "monolith brick." The load carrying capacity was considerably less than that of a single slab having thickness equal to the total thickness of concrete and brick but was about that of a single slab having an "equivalent thickness" conputed by this formula. This may well have been the original use of the formula.

The FAA Advisory Circular[11] specifies that unbonded overlays are used only for a condition factor C_r of 0.75 to 0.35. According to Hutchinson,[12] unbonded rigid overlays for military airfields are used when the existing rigid pavement is badly deteriorated and broken, or when very thick overlays

are used to strengthen thin existing pavements. In such cases, a bond-breaking material isolating the slabs to prevent the base pavements from adversely affecting the performance of the overlay is used. Nonbonded overlays are also used when it is impossible or impractical to make the joints in the overlay coincide with those in the base pavement.

PARTIALLY BONDED OVERLAYS

In a progress report prepared by the Air Transport Division of the American Society of Civil Engineers (ASCE) in 1955[17] the following formula for a portland cement concrete overlay placed directly on the existing concrete slab was first cited:

$$h_o^2 = h_d^{1.87} - C_r h_e^2 \tag{5}$$

where

h_o = thickness of overlay slab, in.

h_d = thickness of equivalent single slab placed directly on the subgrade with a working stress equal to that of the overlay slab

h_e = thickness of existing slab, in.

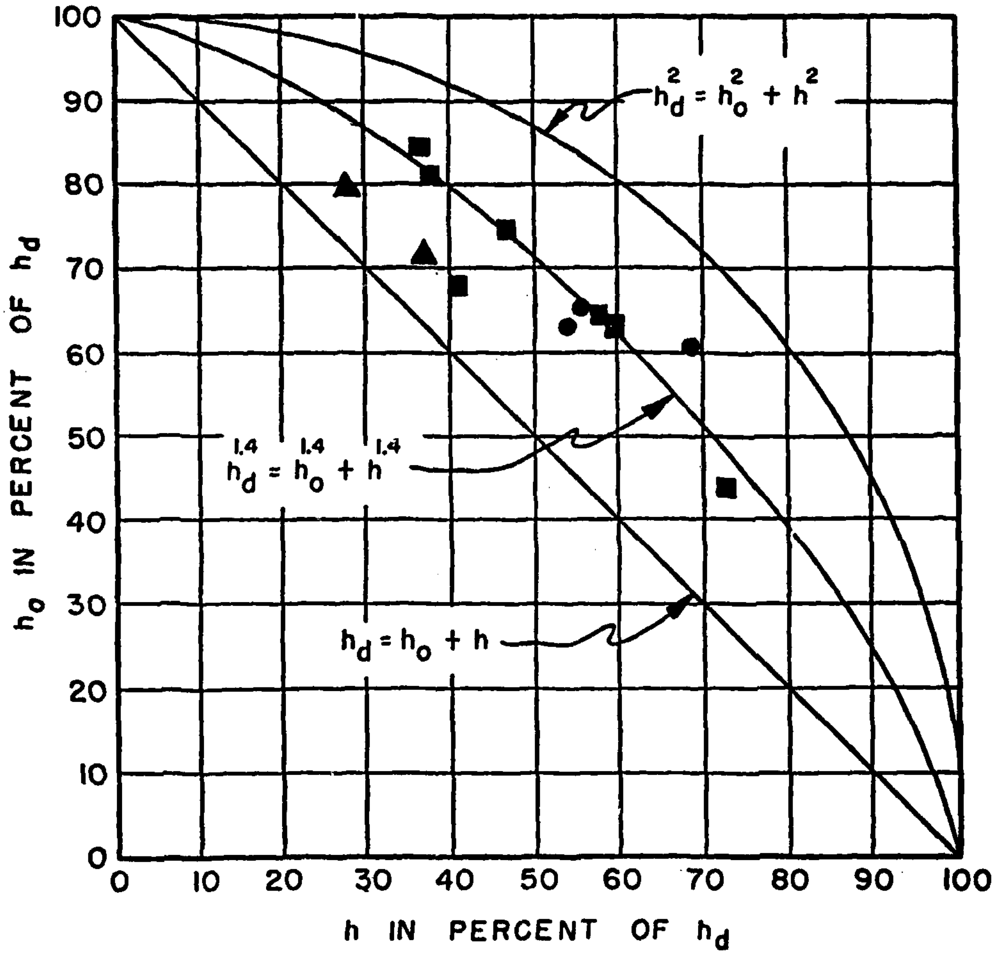
C_r = coefficient, depending on condition of existing pavement

The use of 1.87 in Equation 5 (instead of 2.0 in Equation 4) was claimed to account for the friction between the two slabs. It was recommended that no overlay slab of portland cement concrete be less than 6 in. in thickness. In the late 1950's Equation 5 was further modified to the form now in use for partially bonded overlays:

$$h_o^{1.4} = h_d^{1.4} - C_r h_e^{1.4} \tag{6}$$

The modification was based on the results of Corps of Engineers full-scale traffic tests shown in Figure 1. The coefficient C_r has a value of 0.75 or more. The use of a concrete overlay pavement directly on an existing rigid pavement (without a leveling course) with a C_r value less than 0.75 is not recommended because of the likelihood of reflection cracking.

FAA Advisory Circular[11] specifies that partially bonded overlays are used only for condition factor C_r of 1.0 to 0.75. The use of a concrete overlay pavement directly on an existing rigid pavement with a condition factor of less than 0.75 is not recommended because of the likelihood of reflection cracking.



LEGEND

● – SINGLE WHEEL LOAD
■ – TWIN WHEEL LOAD
▲ – TWIN TANDEM WHEEL LOAD

Figure 1. Development of rigid overlay, partially bonded design criteria

BONDED OVERLAY

Bonded overlays are not used as extensively as partially bonded overlays. Although adequate data were not available for full verification, it was suggested that the design of bonded concrete overlays assume that "an old slab with a bonded resurfacing be treated as having a structural capacity equal in every way to that of a single slab with thickness equal to the total thickness of old slab plus the bonded resurfacing."[16] In other words, the equation for bonded overlay has the form

$$h_o = h_d - h_e \tag{7}$$

FAA Advisory Circular[11] specifies that bonded overlays should be used only when the existing rigid pavement is in good condition. For military airfields,[12] bonded overlays are used only for resurfacing an existing pavement when large increases in load-carrying capacity are not required. Hutchinson[12] gave two reasons that the bonded overlay is used only for resurfacing purposes: first, a method for providing load-transfer devices in the joints of the overlay which will be compatable with the load transfer in the existing pavements has not been developed and proven; and second, when large thicknesses of overlay are required (6 in. or more), it is generally found to be more economical to use the partially bonded overlay method.

SUMMARY OF THE BOARD OF INVESTIGATORS' REPORTS

IDENTIFICATION OF DEFICIENCIES

This section presents a summary of the major deficiencies and some discussions in the existing FAA overlay design procedures. Most of the material presented here is summarized from References 5 through 9. The discussion is divided into 11 separate categories, as follows:

- a. Special remarks on Corps of Engineers' overlay test data.
- b. Overlay design approach.
- c. Equivalent design.
- d. Condition of the existing slabs.
- e. F-factor in bituminous overlay design equation.
- f. Overlay design equations.
- g. Load transfer assumption.
- h. Heavy load deflection/subgrade pressure.
- i. Selection of design flexural strength of concrete.
- j. Selection of subgrade k-value.
- k. *Anomalies of the design* procedure.

SPECIAL REMARKS ON CORPS OF ENGINEERS' OVERLAY TEST DATA

The Corps of Engineers' overlay test data, which were examined by the Board of Investigators, are identified in Appendix A. Two features were built into the test data upon which the FAA design equations for overlays were based. These two features can be easily neglected if the test data are not closely examined. These two features are explained in the following paragraphs.

Conditions of Base Concrete Pavements. In the overlay tests, a majority of the overlay pavements were built on new concrete pavements. The credibility of such a practice was later questioned; Sharonville No. 3 tests (see Appendix A) were thus planned and trafficked. In these tests, overlays were built on old concrete pavements for which the traffic histories were known and the pavement conditions prior to overlay were recorded. It was found that the performance of overlay pavements built on either old or new concrete pavement was nearly the same, provided the base concrete pavement was in fairly good condition. This conclusion implies that as long as the base concrete slab is

structurally sound, the concrete slab is considered to be as good as the concrete overlay.

Nonrigid Overlays Incorporating Unbound Base Courses and Bituminous Bound Surface Courses. In the nonrigid overlay tests, the overlay pavements consisted of either full-depth AC or an AC surface course over an unbound granular base course. Test results indicated little difference in performance between these two types of nonrigid overlays, the overlays incorporating a high-quality base course material having performed as well as a comparable thickness of full-depth AC. In other words, if the design requires a 10-in. AC overlay, an equivalent design can be obtained by using a 4-in. AC surface course and a 6-in. granular base course.

Although the use of granular base courses in nonrigid overlays is not permitted in the FAA Advisory Circular, the Corps of Engineers' nonrigid overlay test data contain a great many of such pavements which were combined with other overlay pavements in the analysis which resulted in the design equation.

Chou[9] suggested that if the test data are to be reanalyzed, decisions should first be made as to whether: (a) different structural coefficients should be assigned to the AC surface course and the unbound base course in the nonrigid overlay test data, and (b) the past traffic history of the base concrete pavement prior to overlay should be considered.

OVERLAY DESIGN APPROACH

The FAA overlay design procedure for both flexible and rigid overlays is based on the "thickness deficiency" concept. A new single slab is designed using the existing foundation as a support. The difference between the new design slab thickness and the existing slab thickness is the thickness deficiency. Pavement layer condition factors are used to modify the existing thickness to reflect deterioration of the existing pavement (coefficient C_b in Equation 1 and C_r in Equation 3) and equivalencies are used to convert concrete overlay thickness into equivalent bituminous concrete thickness (constant 2.5 in Equation 1).

The success of the "thickness deficiency" approach lies in several basic assumptions discussed below. Concerning the procedure in determining the new design thickness, Kilareski and Anderson[7] commented that the FAA overlay design procedure does not stand alone because it is dependent on the design

procedure for new flexible and rigid pavements. The basic assumptions in determining the new design thickness would influence the final thickness of overlay. They further commented that the design procedures are empirical in nature and have no analytical basis for the analysis. Loadings and operations have now increased beyond those considered when the design procedure was developed during the early part of World War II. Since an analytical basis was not used for the original procedure, it is now almost impossible to expand the design to include the heavier aircraft. The same problem will hold true, in the future, if larger aircraft are built and pavements are subjected to larger loadings.

Kilareski and Anderson[7] pointed out another shortcoming of the procedure in the method used to calculate and predict the design loads. The current FAA procedure uses a critical aircraft loading. All other aircraft are equated to this aircraft. This procedure does not take into account the fatigue property of the material as related to the fatiguing loads imposed by the passage of a mixed spectrum of aircraft.

It should be pointed out that a structural design procedure for rigid pavements based on the multi-layered elastic system (BISAR computer program) has been developed recently at the U. S. Army Engineer Waterways Experiment Station.[18] The procedure combines the full-scale traffic test data and field observations with the mechanistic model provided by the layered elastic program and therefore has the analytical basis discussed by Kilareski and Anderson.[7]

Darter and Smith[6] listed seven basic assumptions used in the FAA procedure in determining the new design thickness that could influence the final thickness of overlay, as follows:

a. Maximum edge stress is computed using the Westergaard analysis of a slab loaded at the edge and resting on a dense liquid foundation (existing voids are not considered).

b. The maximum edge stress is decreased by 25 percent to account for load transfer across the joint (actual existing pavement load transfer varies widely).

c. A design aircraft is selected as the aircraft in the mix that requires the greatest pavement thickness. All other aircraft are converted to an equivalent number of annual departures by the design aircraft.

d. The critical stress of the design aircraft is based on the wheel assembly location which generates the highest stress (perpendicular, parallel, or at an angle).

e. Fatigue is accounted for by converting equivalent design aircraft passes to coverages and designing for a projected number of annual departures. The thickness is then adjusted based on a fatigue curve developed by the Corps of Engineers for designs other than 5000 coverages.

f. The foundation support is a direct input in the form of a modulus of subgrade reaction (k-value).

g. The concrete strength is accounted for by dividing the design flexural strength by 1.3 for a pavement to be subjected to 5000 coverages. This also accounts for fatigue for 5000 coverages and any other factors not otherwise considered.

Concerning the assumption that the rigid overlay material and the existing slab material have identical properties (strength, stiffness, fatigue damage, etc.), Darter and Smith[6] commented that although the overlay and base slabs are both concrete, they have different structural characteristics, based on the following two reasons. First, the base slab usually has a much higher flexural strength and stiffness than the new overlay slab, as the base slab is normally many years old. However, the base slab may have some amount of fatigue damage induced by traffic through the years, and second, the base slab has gone through a series of environmental seasons and has stabilized from drying shrinkage, while the new overlay must go through that phase. However, the base concrete slab may also have suffered climatic damage as well.

It should be pointed out that in Reference 17, the authors commented that while the flexural strength of the existing slab does not enter into the computations of overlay Equation 3, a substantial difference in flexural strength in the overlay and base slabs would result in a very small change in thickness. However, no particular test results or references were offered by the authors.

A direct thickness substitution relationship is assumed to exist between the bituminous concrete and portland cement concrete, i.e., 2.5-in. bituminous concerete substitutes for 1-in. portland cement concrete used in Equation 1. However, such a relationship does not exist between the two pavement materials. The most obvious difference is their stiffness and permanent deformation properties. Therefore, theoretically correct material equivalency factors could not be developed to allow for the substitution of an equivalent thickness of one material for the other.

Concerning the proper handling of bonding conditions between the overlay and the base slab, the use of the $n = 1.0$ exponent in Equation 3 for full bond is theoretically correct. However, the use of the $n = 1.4$ exponent for

the partially bonded overlays is established empirically based on limited test data (see Figure 1). For unbonded overlay, the validity of the $n = 2.0$ exponent cannot be verified (see previous discussions).

EQUIVALENT DESIGNS

Darter and Smith[6] stated that "the design of the different overlays should result in equivalent performance so that a fair cost comparison can be conducted and similar performance expected." Through years of field observations on overlay pavements designed by the FAA design procedure, FAA found that the overall performance of rigid overlays is generally better than that of flexible overlays.[19] The following is excerpted from Reference 19.

> Current design procedures do not yield consistent long term performance for different types of overlays on rigid pavements. It has been observed that some overlays deteriorate earlier than expected when required thicknesses are computed using these methods. . . . Updated concepts and procedures are needed to enable airport engineers to determine thicknesses of overlay materials that yield equal life expectancy under generally similar conditions.

The discrepancy in the observed performance between the rigid and flexible overlays is believed to be influenced by the factors discussed in the following paragraphs and comments by Ahlvin[20] concerning this subject follow.

Failure Criteria. The subject of equivalent performance between rigid and flexible pavements has been debated for years.[9] The Corps of Engineers has used different failure criteria for rigid and flexible overlays. The initial crack criterion is used for rigid overlay but the complete failure criterion is used for flexible overlays. In other words, the failure of a concrete pavement overlaid with bituminous concrete is characterized by a crack pattern in the overlay surface approaching the crack pattern that is associated with complete failure of a plain rigid pavement. The rationale behind this criterion is that as long as the bituminous surface is in serviceable condition, it is less important how severe the base concrete pavement has cracked. Because of the nature of the failure mechanism in different pavements, the distress modes in concrete and bituminous overlays may be quite different.[9] For instance, dowel bars can be installed at the joint in the rigid overlay to avoid distress along the joint. In a bituminous overlay, however, reflective cracking can always be found along the joint. Increasing the overlay thickness can minimize reflective cracking along the joint, but

the pavement will definitely be overdesigned elsewhere and consequently increase the total cost.

Because the bituminous concrete overlay can tolerate more cracks and ruts than a concrete overlay, the surface appearance of a bituminous overlay can become worse than that of a concrete overlay. A bituminous overlay may still be functionally serviceable but may "look" very bad due to the cracks and ruts. Chou[9] suggested that before the subject of equivalent design can be discussed, it should first be determined whether the failure criteria for flexible and rigid overlays should be reexamined. Monismith, Yücë, and Finn[5] also emphasized the importance of a unique failure criterion in an overlay design procedure. They stated that the question as to what constitutes failure in the test items is an important question to be answered. Some of the variation in test results may be in part due to differences in what has been designated as failure in the different test tracks. If future tests are planned, it is imperative that the definition of failure be agreed upon in advance.

Darter and Smith[6] also pointed out that different failure criteria were used in the development of the bituminous and concrete overlay equation. The failure of the test sections from which the 2.5 factor was developed (Equation 1) was defined as "the number of traffic coverages corresponding to the first signs of visible deflection."[13] An approximate pavement condition index* (PCI) can be calculated by assuming high severity rutting occurring in the wheel paths. This gives a PCI of 35 to 45. This differs considerably from the definition of failure for the pavements with concrete overlays (Equation 3) which was defined as "one or two structural breaks or cracks occurring in 30 percent of the slabs in a pavement feature, with the cracks starting to spall."[29] An approximate PCI can be calculated by assuming some medium and high severity slab cracking occurring over 30 percent of the slabs. This gives a PCI of 50 to 65. Darter and Smith thus concluded that the resulting design failure condition of the bituminous overlay is worse than the design failure condition of the concrete overlay.

Darter and Smith[6] further illustrated the difficulty, by using numerical computations, of designing "equivalent" pavements using the FAA procedure for overlays. Overlays were designed for an existing 10-in. concrete slab for 6000 annual departures of a DC-10-10 aircraft (450,000-lb gross load). The k-value was 200 pci. An average modulus was used for the bituminous concrete overlays. The designs are given in Tables 1 and 2. The critical stresses in

Table 1

Concrete Overlay Thicknesses (inches) Designed by the FAA Procedure Using Equation 3

Bond	n = 1.0	n = 1.4	n = 2.0
C_r	Full	Partial	None
1.00	4	7	10
0.75	-	9	11
0.35	-	-	13

$h_e = 10$ in. $h = 14$ in. $h_o^n = h_d^n - C_r h_e^n$

Table 2

Bituminous Overlay Thicknesses Designed by the FAA Procedure Using Equation 1

C_b \ F	1.0	0.92	0.85
1.00	10	7	5
0.90	12.5	10	7
0.75	16	13.5	11

$h_e = 10$ in. $h = 14$ in. $t = 2.5\,(Fh - C_b h_e)$

the overlay and base slab were calculated using a finite element computer program ILLISLAB.** The program is described in Reference 30. The joints were assumed to have good load transfer from dowels plus aggregate interlock. The resulting stresses are shown in Table 3 to provide approximately the same critical stress for the different overlays. These indicate that the 2.5

* The development of PCI for both flexible and concrete pavements can be found in References 21-27. WES made a review for the FAA of available condition survey procedures[28] and recommended the PCI for use on civil airports.

** Brief descriptions of the finite element programs ILLISLAB and WESLIQID (developed at WES) can be found in Appendix C.

Table 3

Calculated Critical Stress for Certain Overlay Pavements Shown in Tables 1 and 2

Overlay Type	Overlay Thickness	Bond Type	Critical Base Slab Stress	Critical Overlay Stress
Concrete	4 in.	Full	290 psi	-
Concrete	10 in.	None	261 psi	261 psi
Asphalt	10 in.	Full	288 psi	-

equivalency factor provides for roughly the same stress in the base slab. Therefore, the base slab should experience about the same cracking regardless of the type of overlay. However, Darter and Smith commented that the overall performance of the different overlays may still be considerably different due to other reasons. Two overlay pavements may be structurally equivalent, but quite different functionally.

A study conducted by Shahin and Darter[27] indicated that even where the critical stresses in the base slab were equal, bituminous overlays have not performed as well as either new concrete pavements or concrete pavements with concrete overlays. On the other hand, concrete pavements with concrete overlays have performed as well as the original construction slabs.[25] The data show that the bituminous overlay has a PCI approximately 20 points lower than the concrete overlays over a 20-year period even though the stress in the existing slab is equal.

Bonding Conditions in Concrete Overlays. Darter and Smith[6] stressed the inadequacy of assuming one single exponent value ($n = 1.4$ in Equation 3) for all partially bonded overlays. They felt that the term "partially bonded" should be a "concrete overlay without leveling course" in the current FAA procedure, since the improved performance of this type overlay over an unbonded overlay is derived from the rough interface friction between the two layers. The use of one exponent for all partially bonded pavements assumes that the friction achieved between the two layers is the same for all pavements. In fact, some pavements are worn very smooth, while others have a relatively rough surface before an overlay is placed. The friction and resulting shear

transfer between the slabs will be different for these two conditions. As a result, the performance should be better for the pavement and overlay with the rougher interface.

The inadequate representation of the bonding condition between the overlay concrete and the existing base concrete slab can lead to an incorrect design of concrete overlays. In turn, it can also be an obstacle to achieving an equivalent design using the FAA design procedure.

Accelerated Traffic Tests. The Corps of Engineers' overlay design equations were developed from accelerated traffic tests. Darter and Smith[6] pointed out that one problem with such testing, especially with bituminous materials, is that it does not account for the changes in material properties that occur over time. As the bituminous material oxidizes over time, it hardens and becomes more susceptible to distress. Nonload-associated distress is believed to be occurring in the bituminous overlays which would reduce the overall PCI. Similar distress is however not found for concrete overlays. Therefore, comparisons between bituminous and concrete overlays are not on an equal basis.

The Subjectivity of C_b, C_r, and F Factors. C_b, C_r, and F are used in the overlay design equations to adjust the resulting overlay thicknesses, thus affecting the resulting performance of the overlays. Since the values of the condition factor C_b (for bituminous overlays) and C_r (for rigid overlays) are determined without analytical basis and the selection of the factors is also based on loose and general guidelines, the performances of the resulting bituminous and concrete overlays can hardly be "equivalent." The matter is further complicated by the F factor in the AC overlay that is used to assure a "complete failure" in the existing base concrete slab. The significance of these factors will be discussed later in this report.

Concerning the subject of equivalent design of rigid and flexible overlays, comments by Ahlvin are presented in the following paragraphs:

> Pavement failure is a conceived condition, which rarely, if ever, can be considered as a sudden occurrence or even a condition attained at a particular discernible time. Deterioration and development of distress occur over a period of time with continuing use. We speak of failure as a particular condition and we attempt to quantify some combination of attributes, which have themselves had to be quantified in some fashion, as a particular measure of failure. We need to do this and to continue to perfect the process, but we must not let this lead us to believe in a unique failure point or condition.

There are many considerations which legislate against a unique definition of failure and some are of particular concern in relation to overlay behavior. The elements of distress or deterioration are many and do not impact consistently on pavements of different types, nor on pavements serving different purposes, nor in relation to functional as opposed to structural failure, nor in corrective action necessary to eliminate or compensate for the distress or deterioration. Some elements are observable as cracking, spalling, depressions, pumping, etc., but some are hidden as bottom cracks not yet to the surface, fatigue effects, voids beneath slabs, shearing versus densification in nonrecovering displacement, etc.

In relation to behavior of overlays and in particular in attempting to provide equivalent alternatives of flexible or rigid overlays for design the inconsistent nature of these distress or deterioration elements or combinations of them seem to be at a maximum. Both the actual physical phenomena and their perceived effects on structural behavior are significantly different from flexible and rigid overlays of rigid pavements. Nominally, the same behavior of a base pavement can have led to failure of a rigid overlay while contributing support substantially short of failure of a flexible overlay.

If some single quantitative designation of conditions to be taken as terminal (i.e., failure) is to be developed for use in design and life cycle analysis and for comparable behavior of different overlay types, it will have to be for some single, quite completely defined, set of circumstances with regard to what is expected of the pavement.

Ahlvin[20] concluded that because the extant technology has not prepared pavement engineers to design rigid and flexible overlays on an equivalent basis, the designers should anticipate inconsistencies between flexible and rigid overlay designs. Ahlvin also commented that:

. . .It has been mentioned that applications of the PCI system have shown over a 20-year period that concrete overlays retain a 20 point superiority over bituminous overlays. Both the "back application" necessary for the 20-year period mentioned and the internal consistency of the PCI system itself, which must be accepted, leave some doubt of this superiority. The overlay equations have been widely used in much the same way as used by the FAA both in the United States and worldwide, such that markedly poorer performance of bituminous overlays, in relation to concrete, would surely have been recognized and reported. Unless there is better information confirming the nonequivalent behavior, it seems that the present equations will continue to be used and we should continue

support of efforts to improve selection of C_r, C_b, and F values.

CONDITION OF THE EXISTING SLABS

The condition factors C_b and C_r are used in the bituminous and concrete overlay equation, respectively, to account for slab crack and deterioration visible on pavement surface. C_r varies from 1.0 to 0.35 and the C_b varies from 1.0 to 0.75. The factors are used to increase the overlay thickness either to reduce the severity of reflective cracks in the overlay or to reduce further cracking of the base slab from heavy loads. As discussed previously, the values of the factors were without analytical basis. Darter and Smith[6] raised the question of whether a cracked 10-in. slab would provide the same support and resulting performance as an uncracked reduced-thickness slab of, say, 7.5 in. (for $C_r = 0.75$) or 3.5 in. (for $C_r = 0.35$).

Darter and Smith[6] pointed out that the condition factors do not reflect the true fatigue condition of the pavement, because even if the slab has no surface cracks, it will probably have fatigue cracks in the lower part of the slab. If there is only minor or initial cracking present in the pavement, a factor of 1.0 assumes (falsely) that the life of the existing pavement is not decreased by the fatigue and minute cracking known to exist in such a slab. Darter and Smith[6] criticized the FAA overlay design equation because the fatigue damage in the base slab is not properly accounted for.

As stated earlier, a majority of the Corps of Engineers' overlay test pavements were built on new concrete pavements. It was found that the performance of overlay pavements built on either old or new concrete pavement was nearly the same, provided the base concrete pavement was in good condition. This conclusion implies that even though the base slab is several years old and has been subjected to considerable loading, the amount of fatigue damage existing in the base slab will not significantly affect the overall performance of the overlay pavement.

Darter and Smith[6] advocate the application of the PCI method to evaluate pavement conditions. Besides the shortcoming that the condition factors do not adequately account for fatigue damage which may have occurred in the base slab, Darter and Smith also felt that the condition factors cannot account for the effect of patching and slab replacement. If all cracked slabs were replaced, then the condition factor should logically be adjusted to 1.0 (even though fatigue would not be accounted for in the existing noncracked slabs).

However, no guidance is given on how to adjust the condition factor for the case of a crack where the cracked area has been replaced (partial slab replacement).

F-FACTOR IN BITUMINOUS
OVERLAY DESIGN EQUATION

The F-factor reduces the new slab thickness to ensure that the failure of the base slab is characterized by a crack pattern in the overlay surface approaching the crack pattern associated with complete failure of a plain concrete pavement, i.e., a complete failure in the base slab. Explaining it in another manner, the F-factor indicates that the full required concrete slab thickness is not needed because the underlying slab is allowed to crack and deflect more than a conventional rigid pavement. The F-factor is determined as a function of the annual aircraft departures and the foundation k-value. As the k-value increases, the allowable amount of cracking in the base slab can be increased because it will be less likely for a cracked element to punch into the foundation due to shear failure.

Darter and Smith[6] studied the effect of F-factor on the required bituminous overlay thickness. Various bituminous overlay thicknesses were computed for a 10-in. slab loaded by a DC-10-10 aircraft using the FAA procedure (Equation 1) for various subgrade k-values. The critical stress in the 10-in. base concrete slab was computed using the ILLISLAB computer program. The results are plotted in Figures 2 and 3. One curve in Figure 2 shows the FAA overlay design thickness required considering the F-factor adjustment. The other curve was determined using $F = 1.0$ (no adjustment). The results show a large difference in required overlay thickness for higher k-values. The critical stresses are plotted in Figure 3 for various overlay/k-value combinations for each curve of Figure 2. The critical stress for $F = 1.0$ is relatively constant for a range of overlay thicknesses of 6 to 12 in. The critical stress for the FAA required overlay thickness curve considering the F-factor adjustment shows a much larger change for a range of overlays of 3 to 12 in. Based on the computed results, Darter and Smith concluded that the use of the F-factor results in a large reduction in overlay thickness for higher k-values and that this results in considerably higher stress levels in the base slab. The higher stress will then result in increased slab cracking from heavy loads. The stronger foundation is then supposed to prevent major structural punchouts from the broken slab.

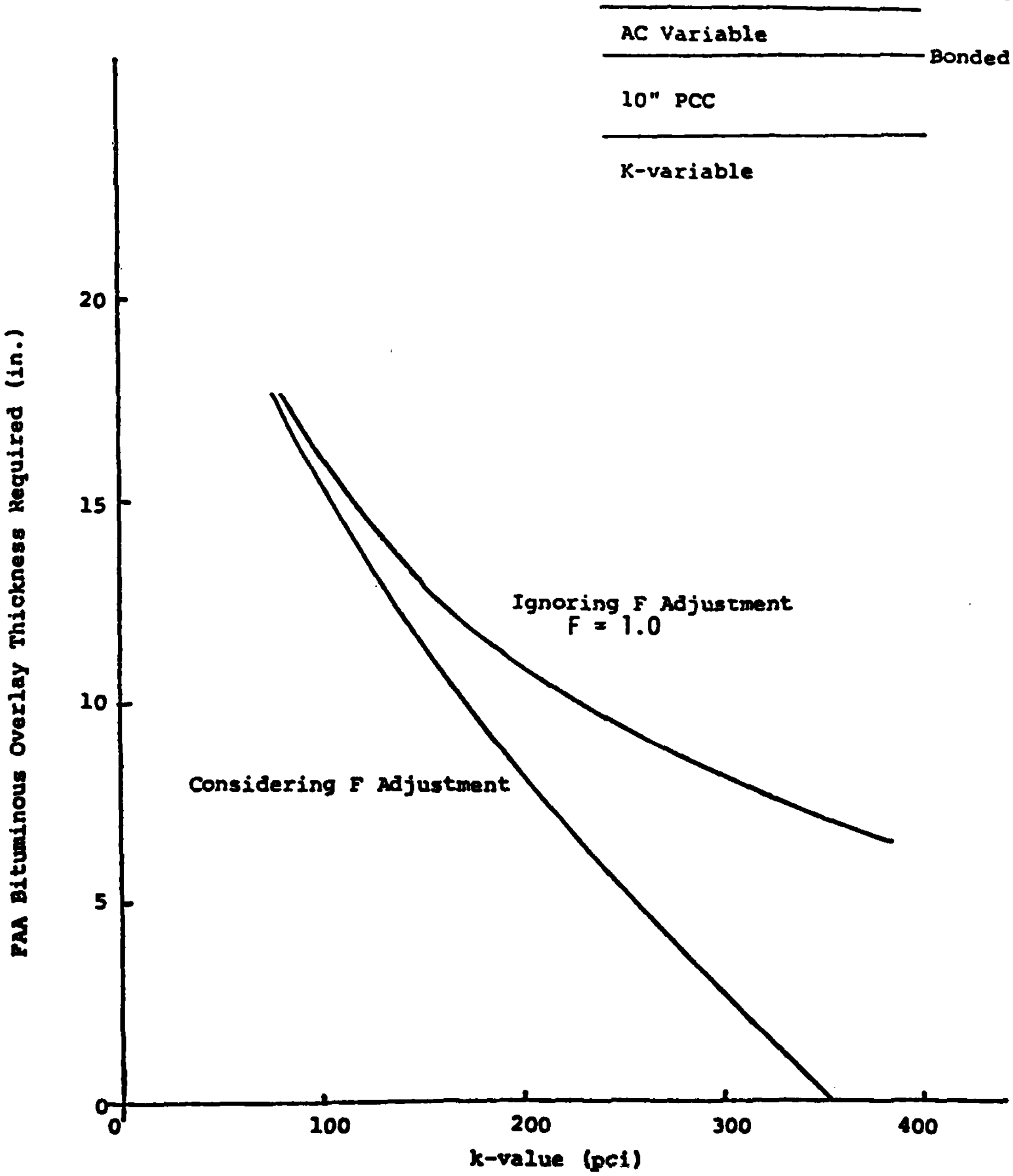


Figure 2. Illustration of effect of F-factor adjustment (after Darter and Smith[6])

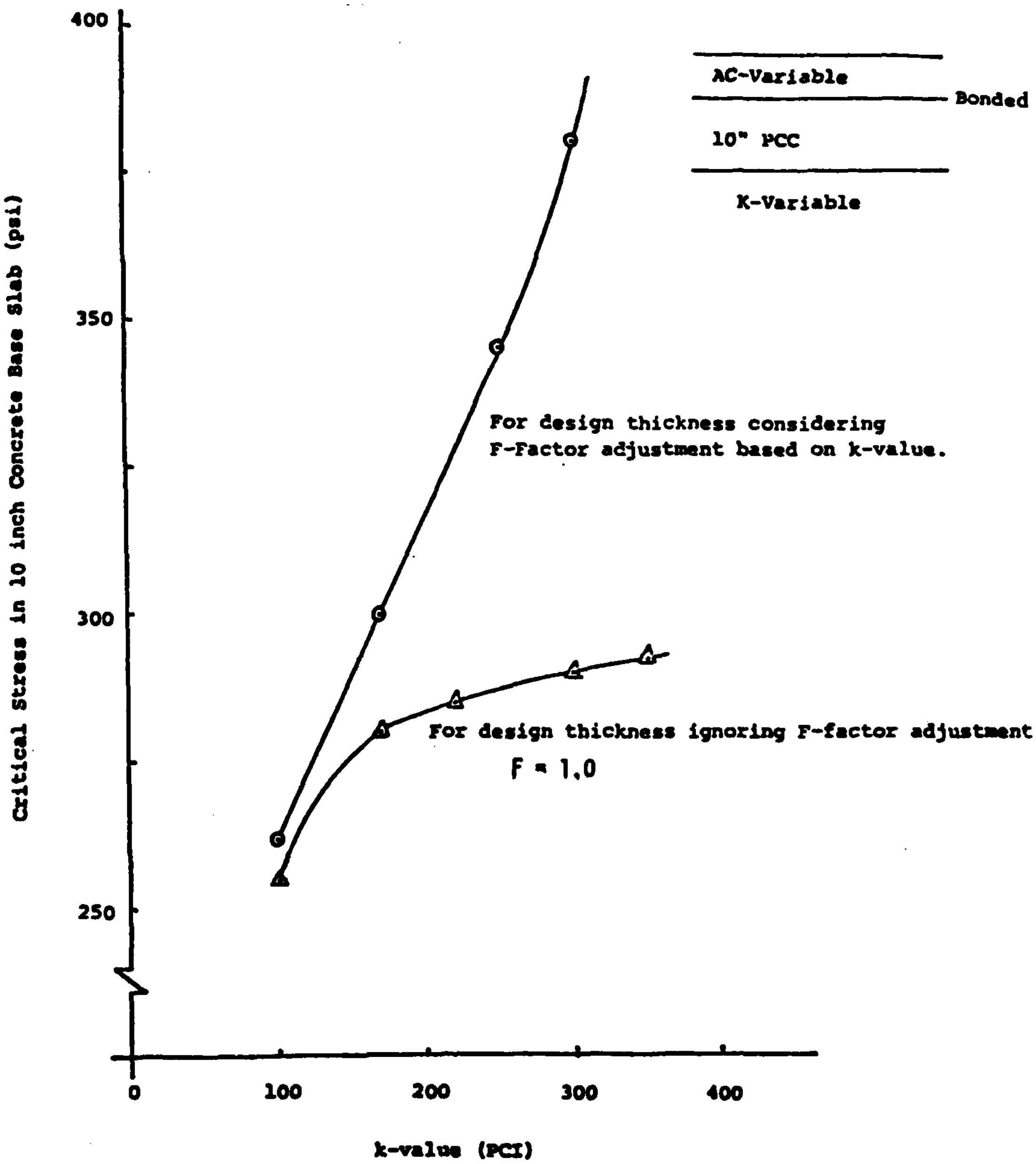


Figure 3. Illustration of effect of F-factor adjustment on stress in basic slab (stresses calculated using ILLISLAB Finite Element Method for DC 10-10 aircraft gear at joint with good load transfer) (from Darter and Smith[6])

Since field experience has shown that the performance of asphalt and concrete overlay pavements designed by the FAA procedure is not equivalent and concrete overlays seem to perform better than bituminous overlays, it is logical that the degree of adjustment of F-factor, which would reduce the bituminous overlay thickness, should be changed to increase the bituminous overlay thickness.

OVERLAY DESIGN EQUATIONS

The structural design equation for concrete overlays of rigid pavements is shown in Equation 3. For fully bonded overlays, if the existing base concrete slab is structurally in good condition and if the fatigue damage in the base concrete slab is so minimal that it does not significantly affect the overall performance of the overlay pavement, the use of $n = 1$ in Equation 3 is correct. The use of $n = 1.4$ for all the partially bonded overlays is too general to expect good results as the value was determined empirically based on Lockbourne test data only (see Figure 1). Since the overall performance of a concrete overlay depends significantly on the bonding condition between the layers and since the interface conditions vary greatly in actual airport pavements, the use of one exponent for all partially bonded pavements is questionable.

For unbonded overlays, the origin of the development of the exponent $n = 2$ is not traceable, as discussed earlier in this report. An attempt was made to use the finite element program WESLIQID (see Appendix C) developed at WES to determine the value of n in the equation for the unbonded case.[9] WESLIQID has the capability of analyzing stress conditions in a concrete overlay; the interface condition can be either bonded or not bonded. The program is similar in nature to ILLISLAB program[30] and is documented in Reference 31. Placing the load at the center of a square concrete slab, computations were made to determine the maximum stresses in the pavement for two different conditions. One condition was for a concrete pavement without overlay (h_d in Equation 3), and the other was for concrete pavements with different thicknesses of overlay (various h_o for a given h_e). The values of h_o were so determined by matching the maximum tensile stresses in the pavements. Preliminary results show that under the no-bond condition, the value of n is in the neighborhood of 2.5, rather than 2.0 as shown in Equation 3. Computations have not been made for other conditions and the value of n may be dependent on other factors not considered.

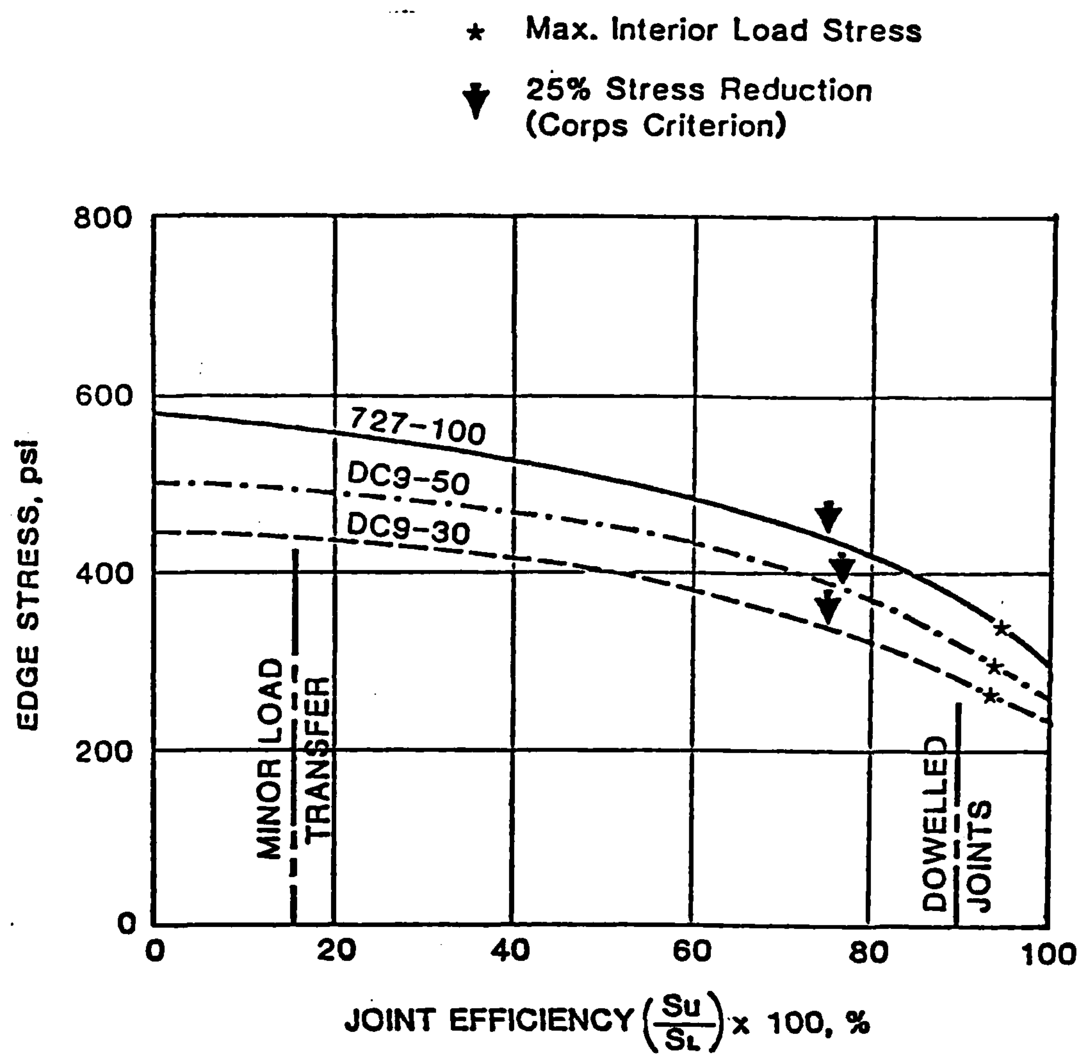
Using the beam-on-elastic foundation theory, Lytton[8] made a theoretical analysis by matching the moment of interia, and found that the exponent n should be 3.0. Although the results obtained by Chou[9] and Lytton[8] are not the same, they both indicate that the exponent n for unbonded overlays may be greater than 2.0. Since test data on concrete overlay with the no-bond condition are scarce, analytical studies of this type of overlay should be pursued further.

In analyzing Corps of Engineers' overlay traffic data, Monismith, Yücë, and Finn[5] concluded that the current criteria may be unconservative and suggested modification to the design equations. Details of the analysis and modifications are presented later in this report.

LOAD-TRANSFER ASSUMPTION

In the FAA design procedure, it is assumed that 25 percent of the load can be transferred across a joint. It is a reasonable approach for new construction, but is questionable for an old pavement that is to be overlaid. The pavement is already in place and the load-transfer capability of the joints may have deteriorated. The load-transfer capability across joints has a dramatic effect on the stress at the bottom of a concrete slab. This change in stress also has a dramatic effect on the performance of concrete pavements. It is relatively simple to measure load transfer across the joint of an existing pavement using heavy load nondestructive deflection equipment currently available. In the FAA design procedure, however, guidance is not provided when the actually measured load transfer is other than 25 percent.

Darter and Smith[6] computed the critical edge stress for various load transfer percentages (or joint efficiency) using the ILLISLAB computer program. The computed results are plotted in Figure 4 for three different aircraft loadings. In the figure, joint efficiency (or deflection load transfer) is defined as the deflection of the unloaded side (S_u) divided by the deflection of the loaded side (S_L) and the result multiplied by 100. The joint efficiency in the FAA procedure would need to be about 75 percent to achieve 25 percent stress reduction. The results plotted in Figure 4 show why the joint area is the critical location at which most slab cracking begins. Experience has also shown that with time, joint load transfer tends to decrease, allowing increased deflection and stress, while center deflection tends to remain nearly constant. Figure 4 shows that the newly constructed pavement has an edge stress of about 380 psi under a B727-100 loading on a 15.5-in. slab.



15 1/2" SLAB
Ec = 4 x 10^6 psi
k = 100 psi

Figure 4. Effect of load transfer on stress in concrete pavement for different aircraft loadings (from Darter and Smith[6])

However, assuming that the load transfer deteriorates to a 30 percent efficiency, the stress in the same pavement will be about 560 psi under the same loading.

The edge stresses computed for the condition of 25 percent stress reduction assumed by the FAA were compared to other levels of load transfer at different slab thicknesses; the differences are plotted in Figure 5. Figure 5 also illustrates the drastic difference in required thickness to achieve a given stress level. For instance, if the required edge stress level were 500 psi for a B727-100 loading, a 12-in. pavement would be needed for a dowelled joint, a 13- to 14-in. slab for the 25 percent stress reduction assumed by FAA, a 15- to 16-in. slab for minor load transfer of about 15 percent, and a slab thicker than 16 in. would be needed if no load transfer exists.

The results presented in Figures 4 and 5 show the importance of joint efficiency in concrete pavements. For instance if the required thickness for a bonded overlay were selected based on the FAA assumed criteria and there were only minor load transfer in the existing pavement, the overlay would be underdesigned by about 2 in. This could seriously decrease the life of the pavement.

HEAVY LOAD DEFLECTION/SUBGRADE PRESSURE

In the FAA design procedures for rigid pavements, the design criteria are based on the maximum stress in the slab with loads placed near an edge. The subjects of deflection and subgrade stress are not considered in the design. It is to be noted that the FAA design procedure was developed long before the heavy wide-bodied jets came into general usage. These heavy loads have been considered in slab stress determination only. However, the large loads are causing large deflections at slab corners, which result in high compressive stresses in the subbase/subgrade. This causes consolidation and loss of support beneath the slab, especially if there is poor load transfer across the slab joints. The end result is corner breaks or diagonal cracks across the slab and serious localized settlement. Recently, Barenberg made an extensive field study at the Chicago's O'Hare International Airport.[32] He found that the majority of the dowel bars near the center of the joint were straight, but that the bars at the corners were badly bent. The cause of the bend was the large deflection at the corner under the heavy aircraft load.

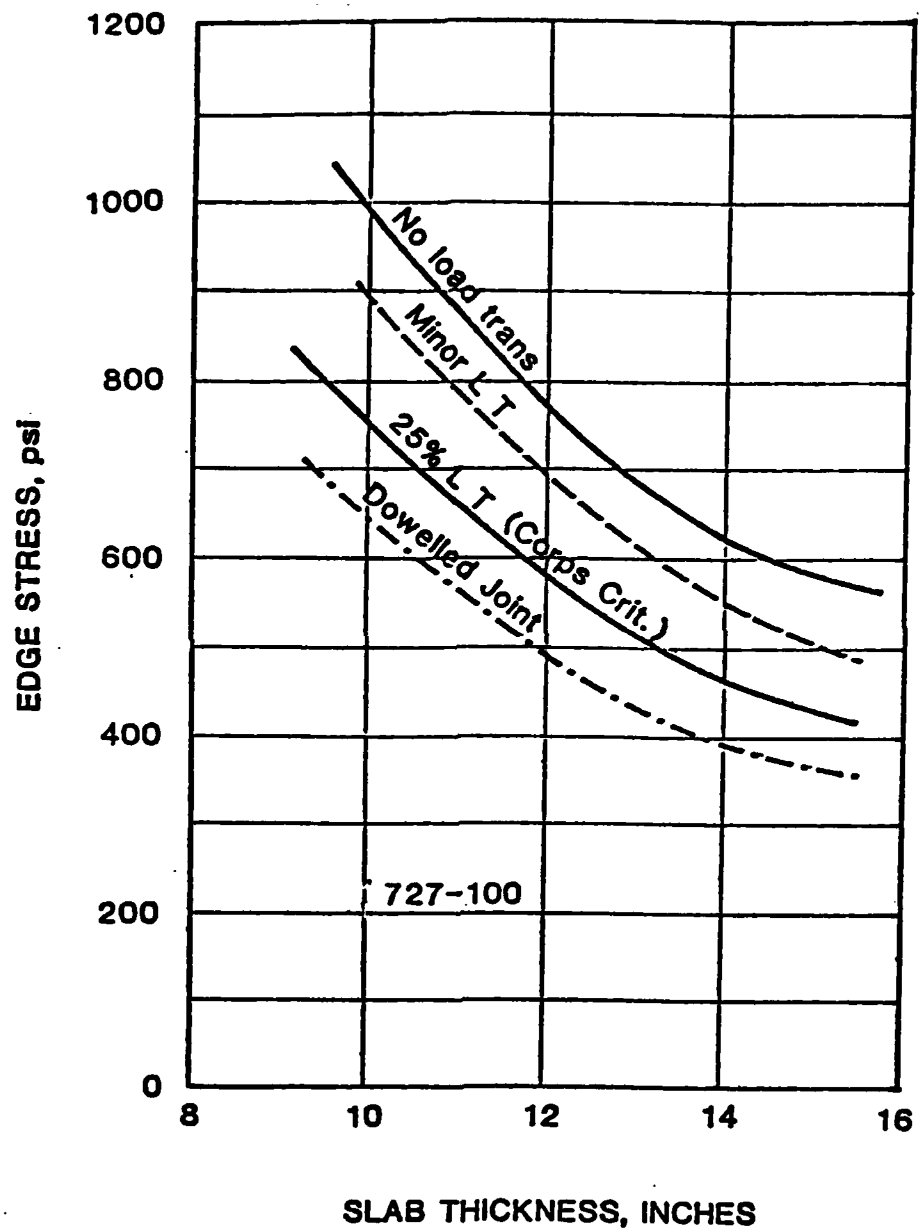


Figure 5. Effect of slab thickness on stress in concrete pavements for different load transfer values (from Darter and Smith[6])

Barenberg suggested the use of deflection criterion in the design of airports that are subjected to heavy aircraft loads.

Computations were made using the WESLIQID finite element program[9,29] to calculate deflections and bending stresses in the concrete slab and subgrade pressures. The square concrete slab was 16 in. thick and had dimensions of 25 ft by 25 ft and was subjected to the Boeing 747F aircraft load. Each wheel of the twin-tandem gear carried a 45,445-lb load. The loads were placed at the corner and at the edge. The distributions of the subgrade pressures are plotted in Figure 6. The maximum bending stresses σ and deflections w are also indicated in the figure.

Figure 6 shows that under the edge loads, the maximum bending stress, deflection, and subgrade pressure were 300 psi, 0.04 in., and 20.5 psi, respectively, in the case of a strong subgrade ($k = 480$ pci). When the k value was reduced to 200 pci, the bending stress and deflection were 481.7 pci and 0.077 in., respectively, and the subgrade pressure was reduced to 15.6 pci. Under the corner loads, Figure 6 shows that for the same subgrade soil $k = 480$ pci, the maximum bending stress was reduced to 287.2 psi but the deflection was increased to 0.093 in. (from 0.04 in.). The reduction in the maximum bending stress is reasonable because edge loading is the critical loading position that produces the most critical stress. The drastic increase in the deflection causes the subgrade pressure to increase from 20.5 to 44 psi. The extremely large subgrade pressure may induce the permanent deformation in the subgrade and consequently create voids at the slab corners. The existence of voids at slab corners indicates possible loss of subgrade support which will result in an increase in the bending stress in the slab and cause the development of corner cracks.

SELECTION OF DESIGN FLEXURAL STRENGTH OF CONCRETE

Darter and Smith[6] stress that in the FAA design procedure there is no guidance given as to the selection of a design flexural strength. For example, two concrete slabs were removed from an airport runway and cut up into standard sized beams. The resulting flexural strength showed a mean of 1043 psi, standard deviation of 200 psi, and range of 650 to 1350 psi. Designers could conceivably select any of the following values (or others):

a. Mean = 1043 psi

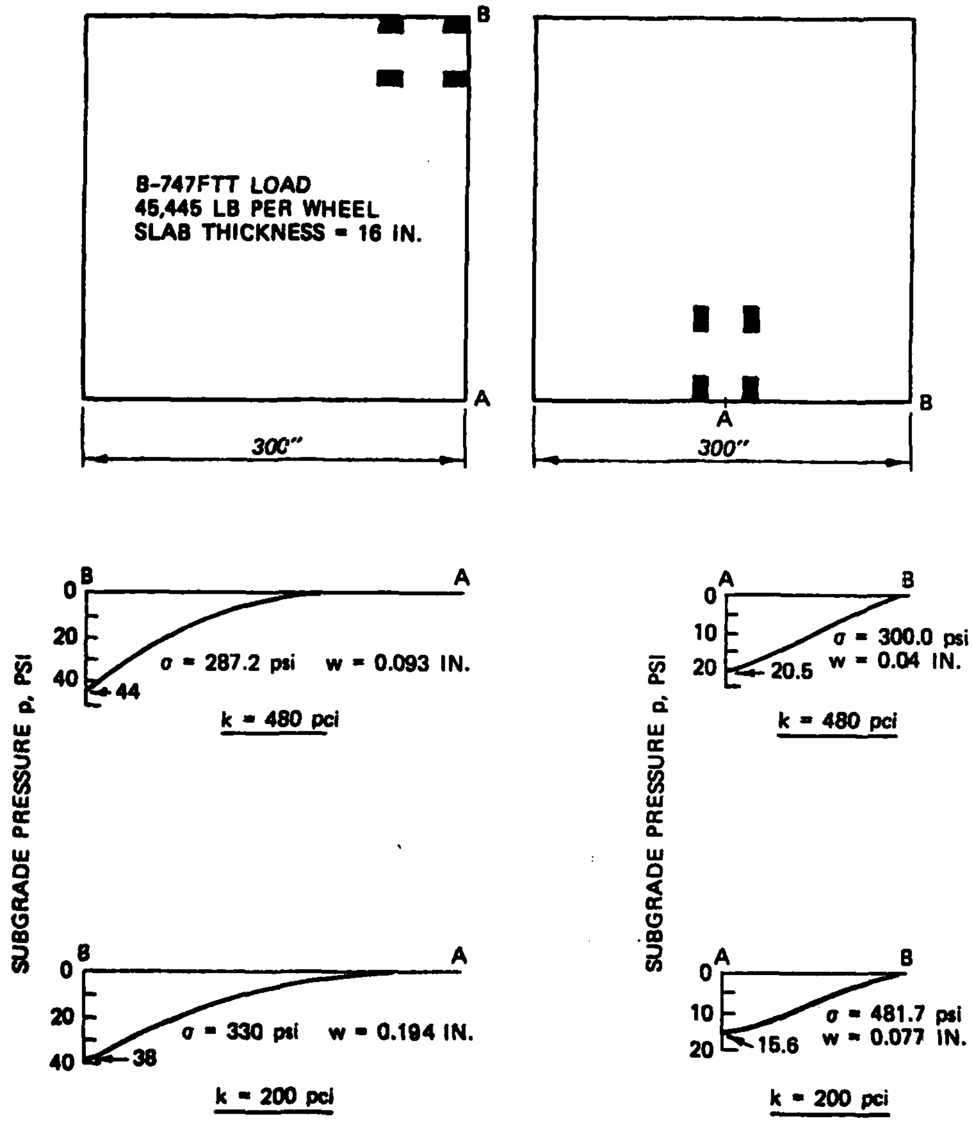


Figure 6. Distributions of subgrade pressures under the edge and corner loads (from Chou[9])

b. Mean - std. dev. = 843 psi

c. Mean - two std. dev. = 643 psi

Each value would result in a greatly different overlay thickness. Guidance as to the appropriate value used for design is not provided in the FAA procedure.

Darter and Smith[6] also pointed out that another aspect of this selection that is not covered in the FAA procedure is whether the flexural strength of the overlay concrete or the flexural strength of the existing base slab should be used in design. The strength of the existing slab is normally higher than that of the new overlay. Actually, this selection should depend on the type of overlay: fully bonded concrete, unbonded, etc.

SELECTION OF SUBGRADE k-VALUE

Darter and Smith[6] pointed out that as with the selection of design flexural strength of concrete, there is no guidance given in the FAA procedure as to the selection of a design subgrade k-value, such as the mean, lowest test value, etc.

ANOMALIES OF THE DESIGN PROCEDURE

Based on the current FAA design procedures, conflicting results may be obtained in some design situations using the concrete and bituminous overlay approaches. Some difference was noticed in the case of strong subgrade soil, but the results are not yet conclusive, and continued study is still needed. It was noted in some cases that the conflict is due to the difference in failure modes between flexible and rigid pavements. This may best be explained by using two example cases.[9]

The first example is illustrated by a hypothetical case. Overlay designs are made for an 8-in.-thick concrete pavement for the C-141 aircraft load which has a gross weight of 350,000 lb. The design is for Type A (channelized) traffic. The design pass level is 100,000 and the flexural strength of the concrete is 600 psi. The design equations for both the concrete and bituminous overlays are

$$t = 2.5(Fh_d - C_b h_e) \text{ for flexible overlay} \qquad (1)$$

$$h_o = \sqrt[1.4]{h_d^{1.4} - C_r h_e^{1.4}} \text{ for rigid overlay} \qquad (6)$$

where

t , h_o = overlay thickness

F = adjusting factor for the flexible overlay

h_d = thickness of the new design

C_b = condition factor for the flexible overlay

h_e = thickness of the existing concrete slab

C_r = condition factor for the rigid overlay

and the exponent 1.4 in Equation 6 indicates that the interface between the overlay and the existing concrete slab is partially bonded.

For different subgrade k-values, the thicknesses of the new design and the flexible and rigid overlays were computed and are tabulated in Table 4. FAA design curves were used to determine the thicknesses of new designs. In computing the thicknesses of the flexible overlays, the F factors were determined based on the subgrade k values and the traffic levels (Figure 7-1, reference 15). The existing concrete slab was assumed to be in good condition, so that C_b and C_r were both equal to 1 in Equations 1 and 6.

Table 4 shows that the design thicknesses for concrete overlays compare reasonably well with those of the new designs. The thickness of the concrete overlays plus base slab are larger than the thickness of new designs. This is reasonable because the interface between the rigid overlay and the existing slab is considered to be partially bonded. However, anomalies exist in the bituminous overlay designs for high subgrade k-values. At k-values of 300 and 400 psi, the bituminous overlay thicknesses are 8.2 and 1.6 in., respectively, as compared with rigid overlay thicknesses of 9.4 and 7.2 in., respectively (for k = 300 and 400 psi). Obviously, the designed thicknesses of the concrete and bituminous overlays for higher k values are not compatible.

Table 4

Comparisons of Overlay Designs with New Designs

Subgrade k, pci	Thickness, in. New Design	Concrete Overlay	Bituminous Overlay
100	19.0	14.8	23.7 (F = 0.92)
200	16.4	11.8	15.3 (F = 0.86)
300	14.3	9.4	8.2 (F = 0.79)
400	12.5	7.2	1.6 (F = 0.69)

The second example is an actual field case. The existing airport runway was a composite pavement having a 12-in. concrete base slab which was overlaid with an 11.5-in. bituminous concrete. The measured k-values of the subgrade range from 180 to 250 pci and the measured k-value at the runway surface was about 480 pci. The runway was to be strengthened to carry the B727-200 (173,000 lb) aircraft for an estimated traffic of 6,000 annual departures. Both bituminous and concrete overlays were considered.

Based upon the strength of the subgrade soil, the current FAA flexible design procedure requires a pavement with a total thickness of 24 in. Since the existing 12-in. concrete pavement could be considered as the base course and the existing 11.5-in. bituminous concrete pavement was in fairly good condition, the existing runway could carry the design aircraft load without overlay. The bituminous overlay can also be designed using the bituminous overlay design equation as suggested in paragraph 67d of the existing FAA Advisory Circular. The required single slab thickness determined from design curves is 16 in. Assuming an F factor of 0.97 and C_b of 0.95, the required thickness of bituminous overlay, as if the existing 11.5 in. overlay were not present, can be calculated as

$$t = 2.5\ (0.97 \times 16 - 0.95 \times 12)$$
$$t = 10.3 \text{ in.}$$

Since the required bituminous overlay is less than the existing 11.5-in. bituminous concrete pavement, again the existing runway could carry the design aircraft without overlay.

To design the concrete overlay, one can consider the existing composite pavement as the subbase and design the concrete overlay as a new concrete pavement based on the measured k-value on the existing runway surface. Based on a concrete flexural strength of 650 psi and the measured k-value of 480 pci, the design curves indicate that a 14-in. PCC was required to carry the aircraft load. The concrete overlay can also be determined using the partially bonded concrete overlay equation and assuming that the existing 11.5-in. bituminous concrete will be removed:

$$h_o = \sqrt[1.4]{16^{1.4} - 0.9 \times 12^{1.4}}$$

$$h_o = 8.3 \text{ in.}$$

The above computations indicate that an obvious difference exists in the design results. A logical explanation for this discrepancy is that flexible and rigid pavements have different failure modes under load. When a "thin" rigid overlay is placed on the existing composite pavement, the bituminous concrete layer tends to deform under the load and cause the overlying PCC pavement to crack. To prevent the concrete overlay from extensive cracking, the design curves indicate that a total thickness of 14 in. of PCC is required. If the existing 11.5-in. bituminous concrete layer is removed, however, the overlay design equation indicates that only 8.3 in. of concrete overlay is needed.

Although both concrete and bituminous overlay design equations are available, engineering judgment should be exercised to determine the best type of overlay for a particular pavement. In the last example, it is obvious that a flexible overlay should be used. When the bituminous overlay design is used, the 11.5-in. bituminous concrete layer (which is indicated as adequate) will rut under the load but this will not constitute failure of the flexible pavement. When the concrete pavement concept is adopted, the supporting k-value is determined at the surface of the composite pavement and the k-value so determined cannot exceed 500 pci. Consequently, the designed concrete overlay should be sufficiently thick to prevent the slab from extensive cracking due to deformation in the supporting foundation which has only a k-value of 500 pci or less. It should be noted that the thickness of the existing composite pavement is not considered in the design of the concrete overlay. In other words, if a very thick composite pavement, say 40 in., is to be overlaid and the measured k-value is actually 800 pci, and in another case a thin composite pavement, say 15 in., is also to be overlaid for the same condition and the measured k-value is 500 pci, the designed concrete overlay thickness will be the same for the two pavements when the rigid pavement concept is used.

SUGGESTED IMPROVEMENTS AND DISCUSSION

The materials presented in the previous section indicate that the existing FAA overlay design procedures have a number of serious deficiencies. Improved design procedures can be developed by (a) modifying the existing procedures, and/or (b) developing completely new procedures using the best available state of knowledge. Darter and Smith[6] commented that since it will take

considerable time to develop sound new mechanistic procedures, it is recommended that certain critical aspects of the existing procedures be improved immediately for use until new overlay design procedures can be developed. In this section, the improvements suggested by the Board of Investigators[5-9] are presented in the following categories:

a. Improvement of existing procedures.
   (1) Consistent determination of C_r and C_b .
   (2) Failure criteria.
   (3) Measurement of actual joint load transfer.
   (4) Guidelines for selection on design flexural strength.
   (5) Use of NDT for determining foundation modulus (k-value).
   (6) Use of a corner deflection criteria for heavy aircraft.
   (7) Improvements of design equations.
   (8) Anomalies of the design procedures.

b. New overlay design procedures.

IMPROVEMENT OF EXISTING PROCEDURES

Consistent Determination of C_r and C_b . Darter and Smith[6] proposed the use of the PCI system to evaluate pavement conditions as PCI is a numerical indicator of pavement condition which is directly related to the pavement's structural integrity and surface operational condition. It is computed as a function of distress type, severity, and amount. Attempts were made to establish a relationship between the condition factors (C_b and C_r) to PCI. Such a tentative relationship was developed for the concrete C_r as shown in Figure 7. Difficulty was encountered, however, for the bituminous concrete C_b for such a relationship. The subjective descriptions used in C_b were studied and it was concluded that for $C_b = 1.0$, the PCI based on structural distress would range from 42 to 78, and when $C_b = 0.75$, the PCI based on structural distress would range from 0 to 100.

Failure Criteria. Chou[9] pointed out that different failure criteria were used in concrete and bituminous overlays, i.e., concrete overlays have been considered failed at the first signs of cracking, while bituminous overlays have been considered failed after considerable cracking and rutting has developed. Darter and Smith[6] stressed that bituminous overlays had a PCI of approximately 20 points lower than the concrete overlay over a 20-year period even though the stress in the underlying (existing) slab is equal.

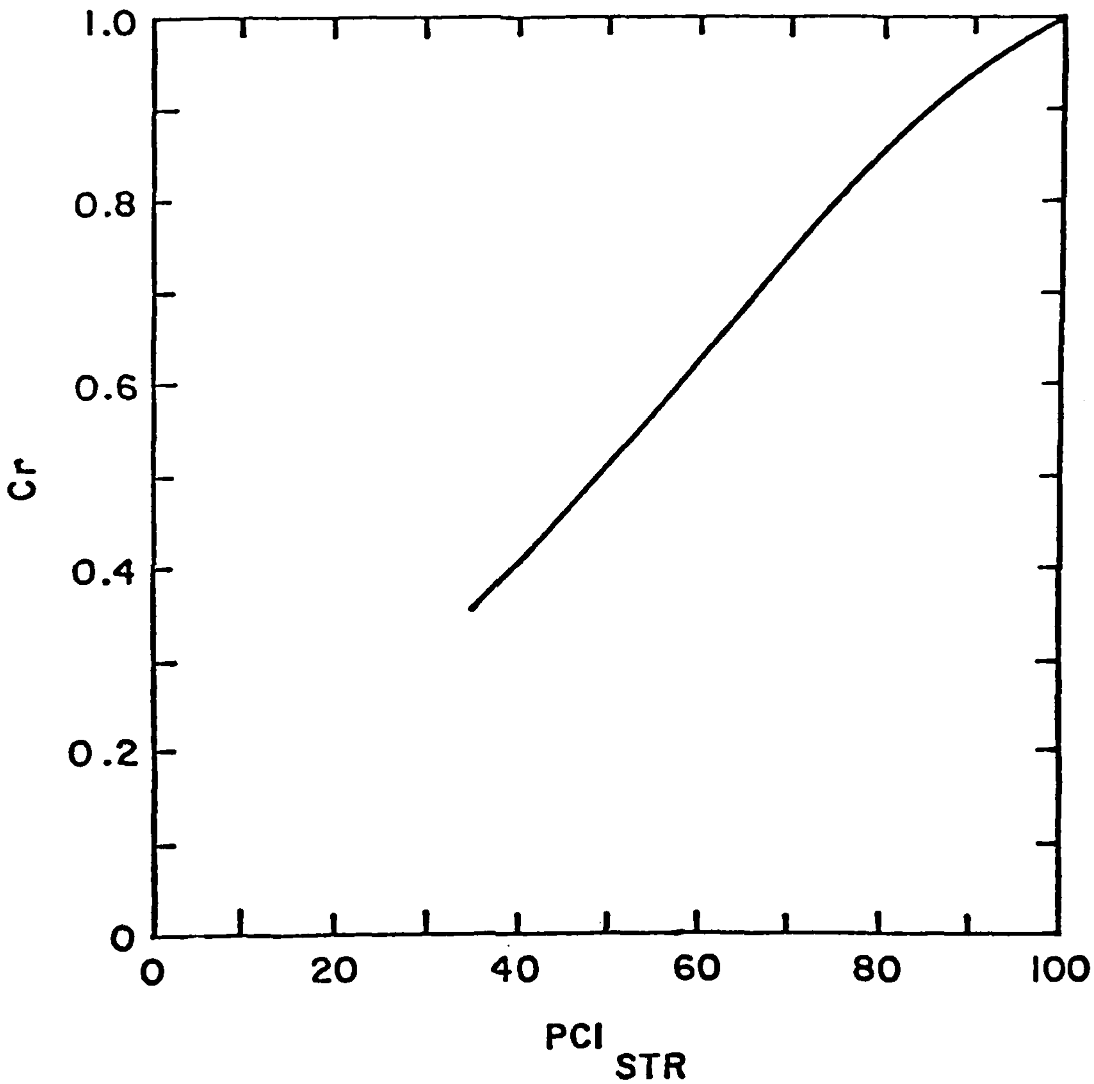


Figure 7. Approximate relationship between C_r and the PCI computed by considering only structural distress (from Darter and Smith[6])

It is evident that the definition of failure in pavement design is very subjective and can hardly be universally agreed upon. If one engineer favors the Corps of Engineers' type of failure criteria, he tends to consider the PCI method inadequate and to feel the method of numerical rating for certain types of pavement distress should be changed. On the other hand, advocates of the PCI method would undoubtedly believe that the use of Corps of Engineers' failure criteria is directly responsible for the nonequivalent design of concrete and bituminous overlays. In the Board of Investigators' meeting held at WES in February 1982, Mr. Ronald Hutchinson and Prof. Carl Monismith both commented that while the PCI method seems to be a good tool for evaluating pavement condition, the method is not capable of describing pavement roughness conditions. The difference of opinion in the definition of failure can further be illustrated by the following discussion.

Tables 5 and 6 present the material types and qualities, measured surface permanent deformations, and coverages to failure of various test pavement sections subjected to multiple-wheel loads. Table 5 is for conventional flexible pavements, and Table 6 is for flexible pavements with stabilized layers. The test pavements were constructed and trafficked at WES. Each test section was generally 50 ft long and sufficiently wide to allow trafficking by either different gear assemblies or the same assembly but with different loads. Although the total load of the Boeing 747 twin-tandem load assembly (240 kips) was less than that of the C-5A 12-wheel load assembly (360 kips), the wheel loads of the twin-tandem were spaced much closer than those of the 12-wheeled assembly and consequently caused greater damage to the pavement. The relationships between the failure coverage and the surface deformation measured at the time of failure are plotted in Figures 8 and 9, respectively, for the conventional flexible pavement group and the group with stabilized layers.

Figure 8 shows that there is a distinct difference in the measured surface deformation in flexible pavements failed by different aircraft loads. The deformations measured at failure for the less demanding load (C-5A) are much less than those for the more demanding load (B747). For instance, at a failure coverage of 280, pavement 7 accumulated a 3.5-in. permanent deformation under the B747 load; and if the pavement were designed for the C-5A load for the same coverage level, the accumulated surface deformation would be only about 1.2 in. Figure 8 also shows that surface deformation at failure increases with an increasing number of coverages (which is proportional to

Table 5

Measured Surface Deformation in Conventional Flexible Pavements, Multiple-Wheel Data

Test Point	Aircraft Type	Assembly Load kips	Thickness in. Surface	Base	Subbase	Subgrade CBR	Coverages at Failure	Maximum Permanent Deformation in.
1	Boeing 747	240	3[a]	6	6	3.7	8[c]	1.8
2	C-5A	360	3[a]	6	15	4	104[c]	0.5
3	Boeing 747	240	3[a]	6	24	3.8	40[c]	2.4
4	C-5A	360	3[a]	6	24	3.8	1500[c]	1.8
5[b]	Boeing 747	240	3[a]	6	24	4	40[c]	2.4
6[b]	C-5A	360	3[a]	6	24	4	1500[c]	1.3
7	Boeing 747	240	3[a]	6	32	4	280[c]	3.5

[a] 4.5 percent AC.

[b] A 3-ft-thick layer of 2 CBR soil placed 21 in. below the subgrade surface.

[c] A flexible pavement item was considered failed when either of the following conditions occurred:

1. Surface upheaval of the pavement adjacent to the traffic lane reached 1 in. or more.
2. Cracking extended through the asphaltic concrete layer.

Table 6

Measured Surface Deformations in Pavements with Flexible Stabilized Layers, Multiple-Wheel Data

				Pavement Description								
Pavement Number	Aircraft Type	Assembly Load kips	Thickness of Surface Course in.	Thickness in.	Material	Stabilizing Agent	Thickness in.	Material	Sub-Stabilizing Agent	Failgrade CBR	Measured Maximum Deformation at to ure in.	Coverages Failure
1a	Boeing 747	200	3	6	Crushed stone	--	24	Lean clay	3% lime 2% portland cement 10% fly ash	5.6	1.2	3660[a]
1b	Boeing 747	240	3	6	Crushed stone	--	24	Lean clay	Same as 1a	4.4	3.2	600[a]
2a	Boeing 747	200	3	0	--	--	25	Lean clay	12% portland cement	5.4	1.1	3660[a]
2b	Boeing 747	240	3	0	--	--	25	Same as 2a		4.0	1.4	340[a]
3a	Boeing 747	200	3	25	Gravelly	5% portland cement	0	--	--	3.8	1.5	7820[a]
3b	Boeing 747	240	3	25	Same as 3a		Same as 3a			3.2	2.5	620[a]
4a	Boeing 747	200	3	25	Clayey sand	5% portland cement	0	--	--	4.9	0.7	1380[a]
4b	Boeing 747	240	3	0	--	Same as 4a		Same as 4a		5.2	1.7	120[a]

a Failure criteria are the same as presented in Table 5.

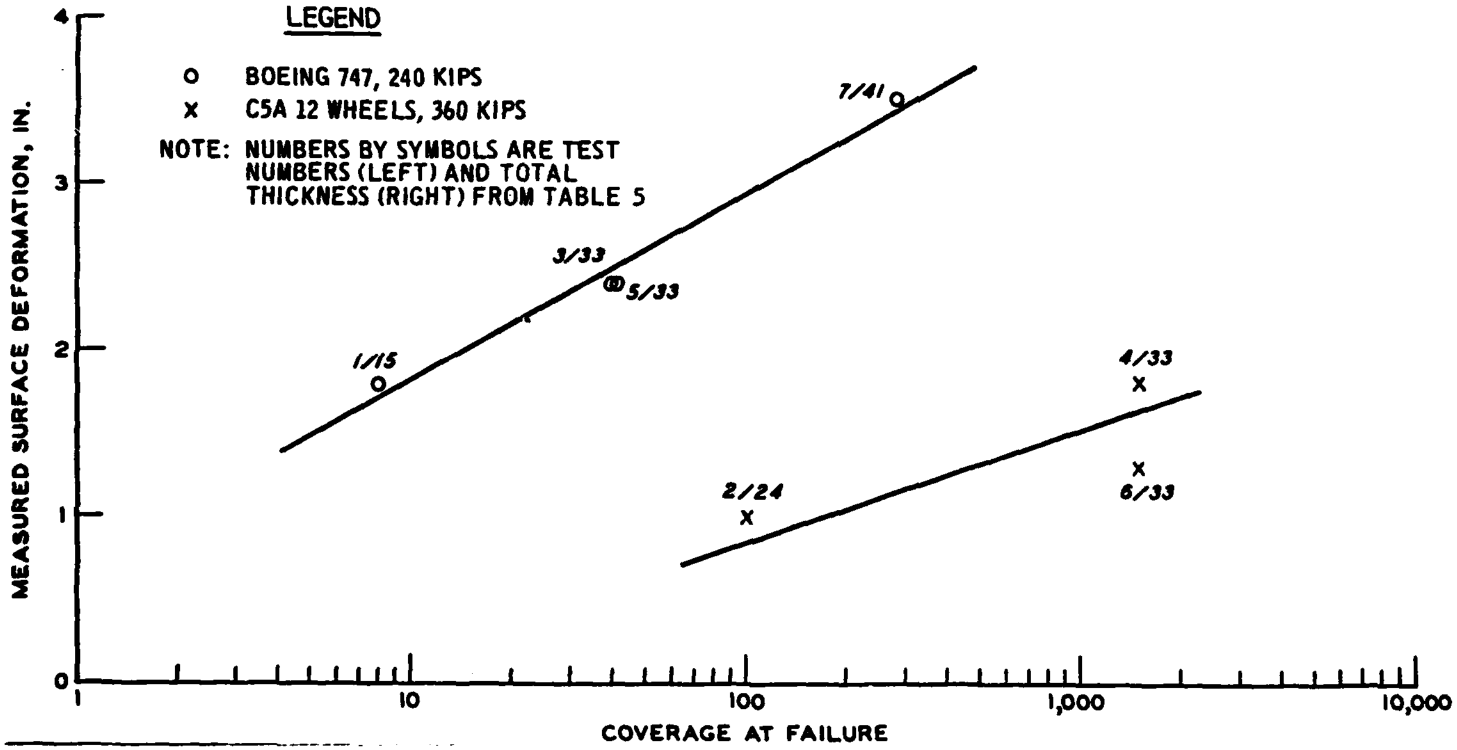


Figure 8. Measured surface permanent deformations of test pavements at time of failure, conventional flexible pavements

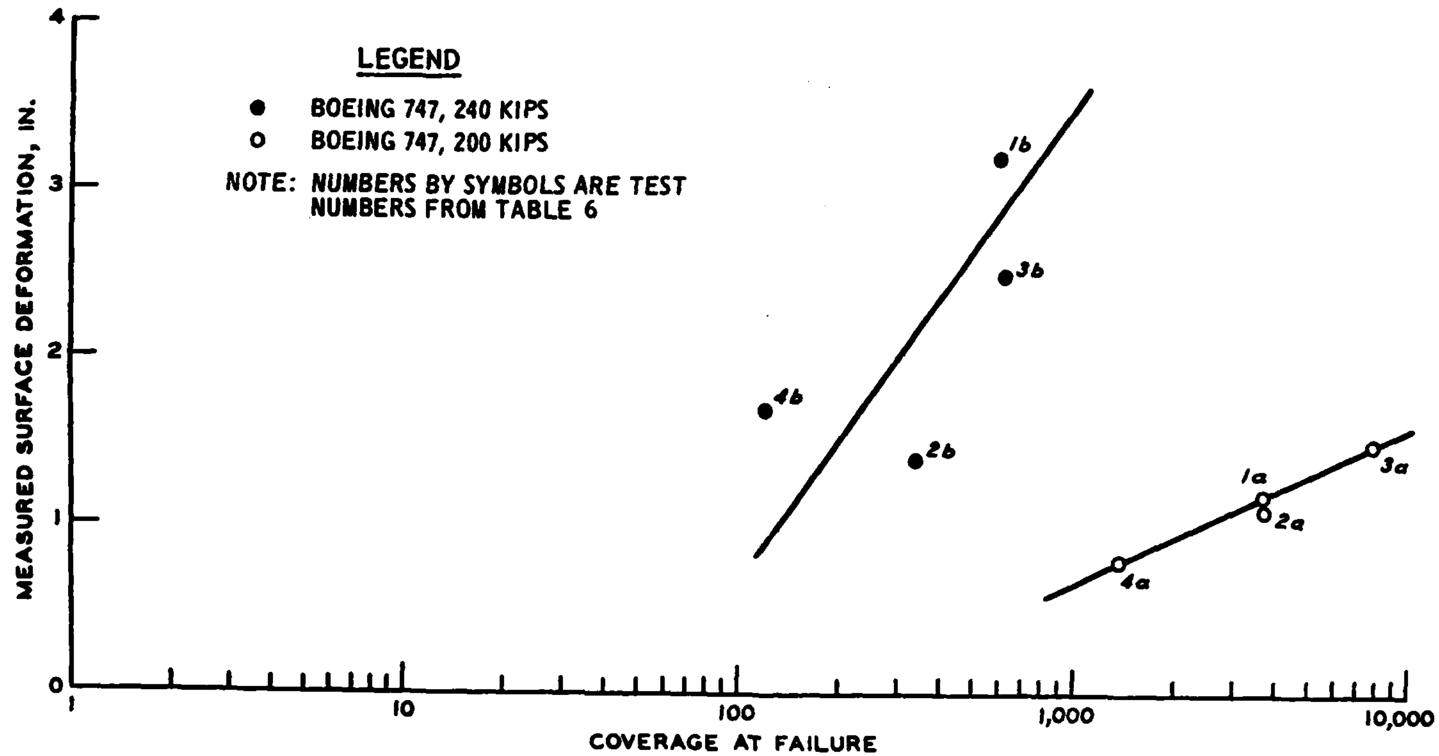


Figure 9. Measured surface permanent deformations of test pavements at time of failure, pavements with stabilized layers

pavement thickness). For instance, under the C-5A load, pavement 1 (15 in. thick) accumulated a 1.8-in. surface deformation at time of failure (8 coverages), but when the thickness of the pavement is increased to 41 in. (pavement 7), the surface permanent deformation becomes 3.5 in. at 280 failure coverages. Figure 9 presents similar results for pavements with stabilized layers. The accumulated surface permanent deformations measured at time of failure for pavements subjected to the 240-kip load were much greater than those subjected to the 200-kip load. Under a given load assembly, the measured surface deformations increase with increasing failure coverage. In other words, two different pavements failed by a given assembly load at different coverages can experience different surface deformations at time of failure, the pavement failed at the higher coverage level has greater measured deformations.

The essential point presented in Figures 8 and 9 is that, based on current Corps of Engineers' flexible pavement failure criteria (see footnote c of Table 5), accumulated surface permanent deformations measured at the time when the pavements are judged failed are not always constant values but vary with many factors, such as gear load, pavement type, and subgrade modulus. The failure criteria, from which failure coverages of field test pavements analyzed were determined, are based primarily on shear failure and cracking of the pavement. Surface rut depth is not considered as a primary factor in judging pavement failure. In fact, surface rutting is not considered a primary factor in judging pavement failure in many existing failure criteria. For instance, the subgrade strain criteria developed by Witczak[33] for full-depth AC pavements were established based on an analysis of field tests conducted by the Corps of Engineers; it is evident that the criterion of surface rut depth is not included in the developed performance model.

In the PCI method, rutting of a flexible pavement is considered to be one of the major distress modes and receives serious consideration in the overall pavement condition evaluation. The severity level of rutting is determined based on the criteria shown in Table 7, and the deduct value of PCI is determined from the measured percent distress density shown in Figure 10. The procedure determining the distress density is explained in references 26 and 27.

The above discussion indicates that different failure criteria are used by the Corps of Engineers and the PCI method. Rutting is seriously considered

Table 7

Mean Rut Depth Criteria

Severity	All Pavement Sections
L (Low)	1/4-1/2 inch
M (Medium)	>1/2 inch but <1 inch
H (High)	>1 inch

in the PCI method but is not considered as a primary factor in judging pavement failure by the Corps of Engineers. Since rut depth greater than 1 in. is categorized as "high" severity in the PCI method, the majority of test pavements in Figures 8 and 9 would have been considered failed at lower failure coverages if the PCI method had been used to evaluate pavement conditions.

Limiting surface rutting is a concept that has been advocated in recent years; however, it has not yet been implemented in many design procedures. Some practicing pavement engineers even believe that if the surface rutting is primarily caused by densification of the pavement materials due to compaction by traffic loadings, and if the pavement materials have no apparent shear failure, the pavement roughness problem is not a serious matter because it can be properly corrected by resurfacing. Structurally, the pavement is stronger than before because of the compaction by the traffic loadings.

The essence of the discussion is to stress the importance of failure criteria employed in judging pavement performance. When a new failure criterion is used to replace an old one, performance data or design equations developed based on the old failure criterion will have to be reinterpreted.

Measurement of Actual Joint Load Transfer. The load transfer is defined to be the ratio of the flexural stress on the unloaded side of the joint to that of the total stresses (the sum of the stresses on both loaded and unloaded sides) in percent. Load transfer across a joint plays an important part in the stresses in a rigid pavement caused by an imposed load. The effect of load transfer on slab stresses was illustrated (using a finite element program) by Darter and Smith[6] (see Figure 11). The figure shows that the critical (maximum) tensile stress in the slab can be greatly reduced when the joint efficiency is increased.

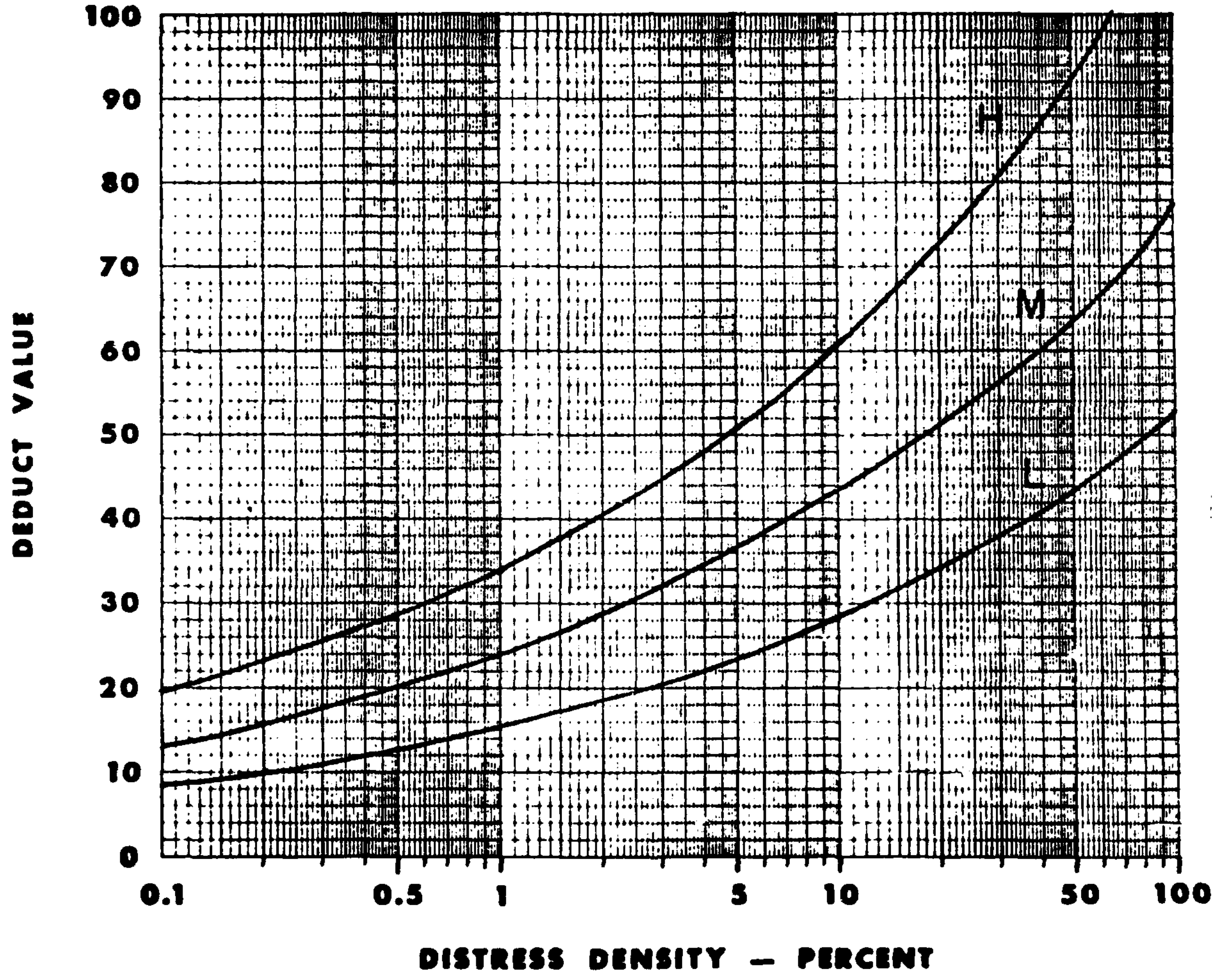


Figure 10. Deduct values of PCI for rutting distress mode (asphalt- or tar-surfaced pavement) (from Shahin, Darter, and Kohn[20])

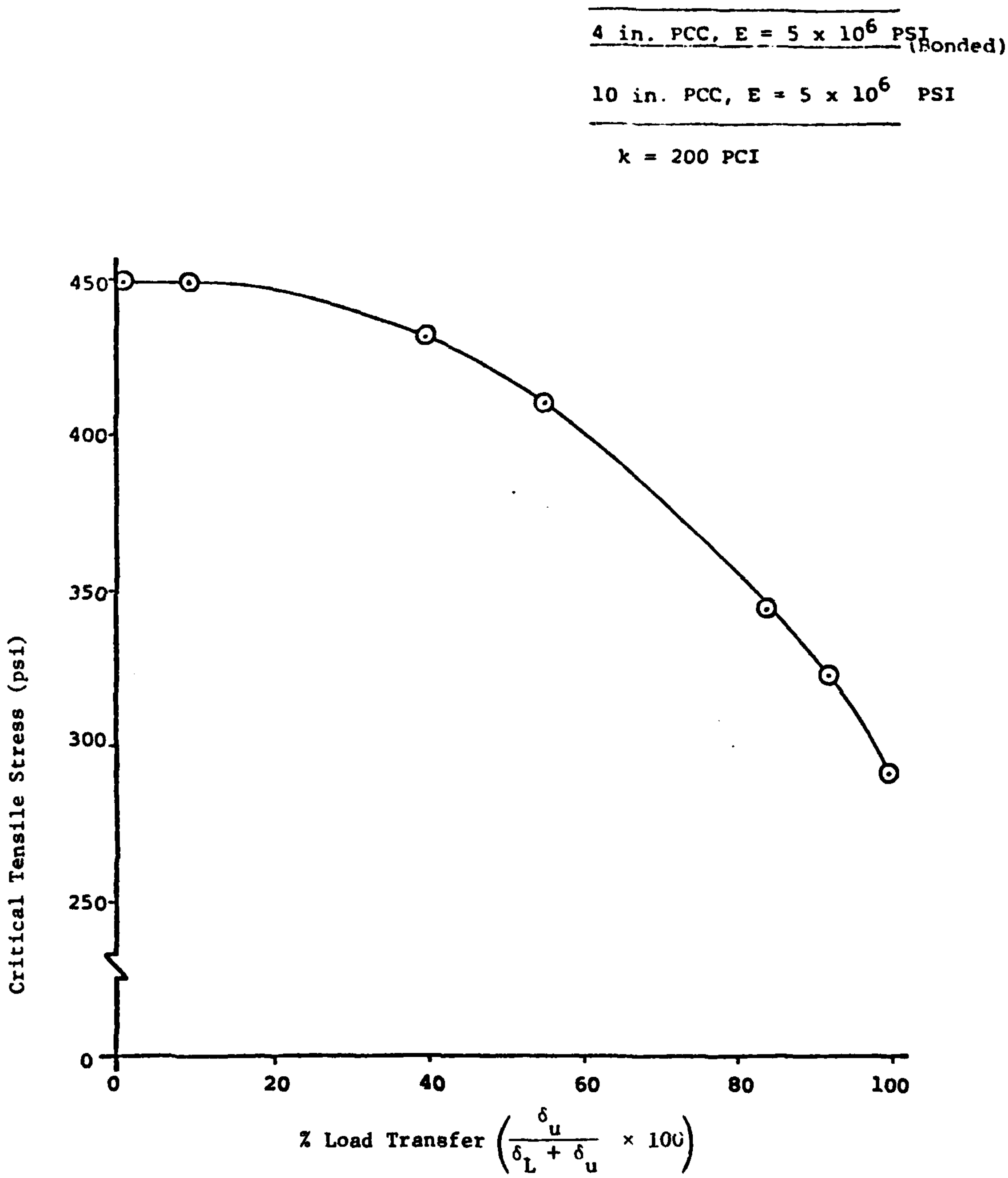


Figure 11. Change in critical stress as affected by load transfer (from Darter and Smith[6])

At WES, two finite-element computer programs are available to analyze the stress conditions in concrete slabs. The WESLIQID program is used to calculate the response of slabs on a liquid foundation, and the WESLAYER program deals with slabs on a layered elastic solid foundation. Several options are available to specify the load-transfer capability along the joint. Table 8 shows the comparison of the load transfers computed by the WESLIQID and the WESLAYER computer programs with those measured in airfield pavements. For instance, the pavements at the Lockbourne Air Force Base were 12 in. thick and the dowel bars were 1 in. in diameter and spaced 15 in. apart. The measured load transfer was 21.2 percent as compared with the 27.0 and 26.5 percent computed by the WESLIQID and WESLAYER programs, respectively.

It is felt that the analytical capacity to simulate field conditions is available. A series of charts giving load transfer ratio as a function of deflection ratios could be developed for different concrete strengths and different foundation supports. This then could be used to modify the required new design slab thickness when it is different from the 25 percent stress reduction currently utilized. Where measured load transfer is considered to be too low, mechanical devices can be installed to improve the load transfer.

Guidelines for Selection of Design Flexural Strength. Darter and Smith[6] proposed the following guidelines to achieve a uniform method:

Type of Overlay	Design Flexural Strength
Bituminous concrete	Existing concrete slab
Fully bonded concrete	Existing concrete slab
Partially bonded concrete	Overlay concrete slab
Unbonded concrete	Overlay concrete slab

This follows the American Concrete Institute (ACI) Committee 325 recommendations and results in logical selection of the strength for the layer where stress will be critical.

The actual value of flexural strength of the existing slab should be determined from either (a) coring the slab at several locations (20 cores minimum) and then testing for indirect tensile strength (and then transposing this strength into flexural strength), or (b) cutting standard sized beams from a few slabs and directly measuring their flexural strength. A standard procedure for selecting beam locations will have to be developed. The mean

Table 8

Comparison of Load Transfer Between Theoretical Solutions and Experimental Measurements

Location	Pavement Thickness in.	Modulus of Subgrade Reaction k pci	Equivalent Subgrade Elastic Modulus E psi	Dowel Diameter in.	Dowel Spacing in.	Load Transfer percent	Subgrade Type	Method
Lockbourne AFB, Ohio	12	75	7,800	1.0	15	21.1	—	Measured
						27.0	Liquid	WESLIQID
						26.5	Elastic	WESLAYER
Lincoln AFB, Nebr.	21	65	7,400	1.5	10	36.8	—	Measured
						27.8	Liquid	WESLIQID
						31.4	Elastic	WESLAYER
Hunter AFB, Ga.	18	175	12,500	1.5	18	27.1	—	Measured
						23.0	Liquid	WESLIQID
						26.9	Elastic	WESLAYER
McCoy AFB, Fla.	18	225	15,000	1.5	18	23.1	—	Measured
						22.0	Liquid	WESLIQID
						26.0	Elastic	WESLAYER
March AFB, Calif.	16	100	8,800	1.5	17	32.5	—	Measured
						28.0	Liquid	WESLIQID
						28.8	Elastic	WESLAYER

and standard deviation of the flexural strengths must then be computed. The design flexural strength can be determined as follows:

$$DFS = MFS - 1.0SD \tag{8}$$

where

DFS = Design flexural strength, psi

MFS = Mean flexural strength from test data, psi

SD = Standard deviation of flexural strength, psi

This provides for approximately an 85 percent level of confidence that the concrete strength will equal or exceed the DFS.

Use of NDT to Determine Foundation Modulus (k-Value). Darter and Smith[6] suggested the use of heavy load deflection devices to determine a k-value beneath the concrete slab. Stiffness measurements should be made at the center of the slab during times when the temperature gradient is approximately zero through the slab. Such measurements have been made on several airfields and highway pavements and reasonable results have been obtained as long as a heavy load is utilized. The k-value obtained is actually an elastic repeated load value that could be reduced to the gross k-value currently specified if further research data show a significant difference in the two values.

The Westergaard deflection equation or finite element programs can be used to develop charts for easy determination of k-value from deflection. This procedure would reduce costs and time and actually provide more reliable foundation support data. Many more tests could be conducted, and the variation of slab support over the pavement feature can be determined.

Use of a Corner Deflection Criteria for Heavy Aircrafts. Darter and Smith[6] proposed the use of deflection criteria to limit the corner deflection under heavy aircraft gears. The use of finite element techniques that adequately model load transfer may provide an analytical procedure. However, they cautioned that this will require a well thought out research study.

Improvements of Design Equations. Monismith, Yücë and Finn[5] reanalyzed the traffic data provided by the Corps of Engineers (see Appendix A). They concluded that the current criteria may be unconservative and suggested modification to the design equations. Their analysis is presented in this section.

Sixty-eight items from ten different traffic tests were investigated. The H-51[34] computer program was used to determine the critical (maximum) edge stresses σ_e of each test item. (The program is prepared based on the Westergaard's solution of a plate on a liquid foundation.) The Design Factor (DF) and Stress Ratio (σ/R) were determined for each item using the following expressions:

$$\text{Design Factor} = \frac{R}{0.75\sigma_e} \tag{9}$$

$$\text{Stress Ratio} = \frac{\sigma_e}{R} \tag{10}$$

where

R = flexural strength of concrete, psi

σ_e = maximum edge stress (tensile) determined by the computer program H-51, psi

The equivalent plain rigid pavement design thickness (or the single thickness of rigid pavement required for design conditions) h_d was computed for each test item based on Equations 1 and 3 for bituminous and concrete overlays. The equations are

$$h_d = \frac{1}{F}(0.4t + Ch_e) \tag{11}$$

$$h_d^n = h_o^n + Ch_e^n \tag{12}$$

where

h_d = equivalent plain pavement design thickness

t = total thickness of flexible (nonrigid) overlay

h_e = thickness of existing rigid pavement

h_o = thickness of rigid overlay

F and C = defined in Equations 1 and 3; $C = C_b$ and $C = C_r$ for cases of bituminous and concrete overlays, respectively

Table 9 contains a summary of the design and actual thicknesses of the pavement components, modulus of subgrade reaction, average flexural strengths of concrete, coverages at initial cracking and at failure, F factors, and computed equivalent plain rigid pavement thickness for each of the 68 test items. Of these 68 items, 19 were discounted:

Table 9

Pavement and Bonding Information of Test

Items Used in the Analyses

SHARONVILLE TRAFFIC TEST NO. 3

25,000 lb load on single wheel

p = 200 psi

Item No.	Concrete Base Design Thickness (inches)	Flexible Pavement Design Overlay Thickness (inches)		Total Design Thickness "h" (inches)	Actual Field Thicknesses (inches)			"k" (lb/in^2)	"R" Average Flexural Strength of PCC Base (lb/in^2)	Coverages at		"F" Factor for A-Type Traffic Area	Equivalent Rigid Pavement Design Thickness "h_d" (inches)
		Binder Course	Wearing Course		Concrete Base	Overlay "t"	Total			Initial Cracking	Failure		$h_d = \frac{1}{F}(0.4t + Ch_e)$
-6	6	2.5	1.5	10	5.8	4.4	10.2	100	950	122	550	0.940	6.50 in.
-7	6	1.5	1.5	9	5.8	3.4	9.2	125	950	122	400	0.925	6.18 in.
-8	6	2.5	1.5	10	5.8	4.6	10.4	125	950	122	700	0.925	6.69 in.
-9	6	3.5	1.5	11	5.8	6.0	11.8	125	950	600	1,600	0.925	7.30 in.
10	6	4.5	1.5	12	5.8	6.7	12.5	80	950	240	East Lane 508; West Lane 1,600	0.955	7.36 in.

SHARONVILLE HEAVY LOAD TEST TRACK

62.75"

31.75"

325,000 lb load on twin tandem gear
Area = 267 sq. inch.
a = 31.25"
c = 62.75"

Item No.	Components of Pavements (inches)			Total Thickness (inches)	"k" pci	"R" Average Flexural Strength (lb/in^2)	Coverages at		Equivalent Rigid Pavement Design Thickness "h_c", inches		
	PCC Base	A. C. Overlay	PCC Overlay				Initial Cracking	Failure	$h_d = h_r + Ch_e$	$h_d = \sqrt{h_r^2 + Ch_e^2}$	$h_d = h_r + 0.3 Ch_e$
68	12	8	18	38	100	687	1,100	8,280		26.91"	24.0"
69	17	-	15	32	100	692	180	4,040		22.67"	21.50"
70	17	-	11	28	100	712	1,970	7,090	28"		

1. R values given in Column 7 are the average of eight test beams for 90 days. Since the difference between the average R values for PCC base and overlay is less than 100 psi, no correction factor is used in equation

$$h_d = \sqrt{h_r^2 + Ch_e^2}$$

2. Cement grouting is applied for Item 70 prior to concrete overlay. That is, a full bond situation exists.
3. In Item 69 PCC overlay is paved directly over the curing compound.
4. In Item 68, 8 inches asphalt concrete laid between the portland cement concrete base and overlay. In the computations the "no bond" equation is used, taking the thickness of existing base as 20 inches.
5. C = 1.0 is used in the evaluation, since the existing basement slab was not subjected to any load. No cracks.

SHARONVILLE TRAFFIC TEST No. 1

18.75"

P = 100,000 lb on twin-wheels

Contact area/tire = 267 in^2.

37.5"

Item No.	Concrete Base Design Thickness (inches)	Flexible Pavement Design Overlay Thickness (inches)		Total Design Thickness "h" (inches)	Actual Field Thicknesses (inches)			Type of Overlay Base	"k" lb/in^3		"R" Average Flexural Strength of PCC Base (lb/in^2)	Coverages at		"F" Factor for A-type Traffic Area	Equivalent Rigid Pavement Design Thickness "h_d" (inches)
		Base	Wearing and Surface		Concrete Base	Overlay "t"	Total		Plate Bending Corrected	Saturation Corrected		Initial Cracking	Failure		$h_d = \frac{1}{F}(0.4t + Ch_e)$
1	6	9	3	18	5.19	12.12	17.31	WBM[b]		110	725[d]		130	0.935	9.35 in.
2	6	15	3	24	5.88	18.00	23.88	WBM[b]		105	725		950	0.930	12.38 in.
3	6	21	3	30	5.63	24.50	30.13	WBM		115	725		> 3,000	0.932	15.05 in.
4	6	12	3	21	5.78	15.75	21.53	AC[c]		75	725		250	0.958	11.10 in.
5	6	9	4	19	5.76	13.19	18.95	AC[c]		75	725		300	0.958	10.02 in.
6	8	9	2	19	8.05	12.15	20.20	AC		75	725		270	0.963	11.29 in.
7	10	6	3	19	9.64	10.28	19.92	AC		75	725		1,250	0.958	11.84 in.
8	6	6	3	15	5.36	9.81	15.37	AC		71	725		130	0.960	8.43 in.
9	8	3	4	15	7.73	8.16	15.89	AC		90	725		485	0.948	9.56 in.
10	10	3	3	16	9.50	6.38	15.88	AC		130	725		> 3,000	0.923	10.48 in.
11	6	3	3	12	6.05	6.09	12.14	AC		120	725		70	0.920	7.31 in.
12	10	-	3	13	9.30	2.88	13.2	AC		100	725		1,150	0.940	8.81 in.
13	8	-	3	11	8.00	2.88	10.88	AC		100	725		70	0.940	7.61 in.
14	6	-	3	9	6.35	3.36	9.91	AC		125	725		60	0.923	6.76 in.
15[a]	10	-		10	9.64	-	9.64			140	725		1,200	-	10.00 in.
16[a]	8	-		8	8.13	-	8.13			160	725		1,600	-	8.30 in.

a. They were initially unsurfaced, but after 1,200 and 1,440 coverages were overlayed with 3-inch AC surface.
b. Waterbound macadam.
c. Asphalt concrete.
d. Average of seven beams. Individual "R" values for each item are not given.

(Sheet 1 of 4)

Table 9 (Continued)

SHARONVILLE TRAFFIC TEST No. 2

P = 100,000 lb on twin wheels

Contact area/tire = 267 sq. in.

37.5"

Item No.	PCC Base Design Thickness (inches)	Flexible Pavement Design Overlay Thickness (inches): Aggregate Base	Wearing and Surface Courses	Total "t"	Rigid Pavement Design Overlay Thickness (inches) h_r	Total Existing Pavement Thickness (inches)	"k" lb/in³	"R" Average Flexural Strength (lb/in²): PCC Base	PCC Overlay	Coverages at: Initial Cracking	Failure	"F" Factor for A-type Traffic Area	Equivalent Rigid Pavement Design Thickness "h" inches: $h_d = \frac{1}{F}(0.4t + Ch_e)$	$h_d = \sqrt{h_r^2 + Ch_e^2}$	$h_d = h_r + 0.3Ch_e$
17	6	31	4	35	-	41	125	775			> 3,000	0.925	20.00"		
18	9	14	4	18	-	26	125	775			> 6,300	0.925	14.27"		
19	6	10	4	14	-	22	125	775			1,135	0.925	12.56"		
20	10	6	4	10	-	20	125	775			1,000	0.925	12.43"		
21	6	-	-	-	16	22	125	775	840		Not mentioned	-		16.39"	16.63"
22	8	-	-	-	15	23	125	775	840		Not mentioned	-		15.72"	15.84"
23	5.75ª	-	-	-	13.25ª	19	125	775	840		> 22,000	-		13.68"	13.85"
24	7.75ª	-	-	-	12.15ª	20	125	775	840		> 22,000	-		13.08"	13.06"
25	9.75ª	-	-	-	10.25ª	20	125	775	840		18,500	-		11.76"	11.17"
26	6	-	-	-	10	16	125	775	840		1,200	-		10.61"	10.63"
27	8	-	-	-	9	17	125	775	840		250	-		10.17"	9.84"
28	10	-	-	-	6	16	125	775	840		230	-		8.43"	7.05"
29	6	38	4	42	-	48	125	775			> 3,000	0.925	23.02"		
30	8	20	4	24	-	32	125	775			> 3,000	0.925	16.86"		
31	6	-	20	20	-	26	125	800			4,000	0.925	13.51"		
32	6	-	14	14	-	20	125	800			1,280	0.925	10.92"		
33	8	-	12	12	-	20	125	800			1,700	0.925	11.68"		
34	6	-	10	10	-	16	125	675			90	0.925	9.19"		
35	10	-	10	10	-	20	125	650			950	0.925	12.32"		
36	8	-	9	9	-	17	125	650			215	0.925	10.38"		
37	10	-	7	7	-	17	125	650			425	0.925	11.16"		
38	8	-	6	6	-	14	125	750			170	0.925	9.08"		
39	10	-	4	4	-	14	125	825			690	0.925	9.84"		
40	12	-	4	6	-	18	125	825			4,620	0.925	11.46"		
41	12	-	3	3	-	15	125	825			2,130	0.925	11.03"		

a. Actual thicknesses.

Note:

1. Since the difference between the flexural strength of existing pcc base pavement and pcc overlay is less than 100 psi, no correction factor is used in equation $h_d = \sqrt{h_r^2 + Ch_e^2}$.
2. "R" value for items 23, 24, and 25 are given. For all other items, soil strength is expressed in terms of CBR.
3. The subsurface explorations through items 17 to 41 show that the subgrade soil falls in the CH group according to the Casagrande Soil Classification as used by the Corps of Engineers. Thus the appropriate "k" value for all items is selected as k = 125 lb/in³ using the charts developed by the Portland Cement Association for CBR = 4 and soil classification CH.
4. A tack coat of RC-2 is applied between ALL CONCRETE and ASPHALT layers.

LOCKBOURNE TRAFFIC TESTS NO. 2

150,000 lb wheel load

Contact area = 1,459 sq. inch; 103 psi contact pressure.

Item No.	Components of Pavement Thickness: PCC Base Slab	PCC Middle Slab	PCC Top Slab	Total Thickness (inches)	"k" pci	"R" Average Flexural Strength (lb/in²)	Coverages at: Initial Cracking	Failure	Equivalent Rigid Pavement Design Thicknesses "h_d" (inches): $h_d = \sqrt{h_r^2 + Ch_e^2}$	$h_d = h_r + 0.3\,Ch_e$	$h_d = \left[h_r^{1.4} + Ch_e^{1.4}\right]^{1/1.4}$	$h_d = h_r + 0.6\,Ch_e$
F12 14-100	10	-	14	24	135	705	10_c	1,000	17.20"	17.00"		
G12 14-100	10	-	14	24	160	840	887_c	1,430			19.80"	20.00"
H12 7-7-100	10	7	7	24	106	740	83_c	1,715			20.30" (17.61")	17.20"
J12 7-7-100	10	7	7	24	215	785	10_{T-C}	370	18.30" (14.07")	17.00"		
L14 14-80	8	-	14	22	220	635	6_c	1,000	16.12"	16.40"		
M14 14-80	8	-	14	22	175	785	36_c	867	16.12"	16.40"		

1. Overlay slabs L14. 14-80, M14.14-80, F12.14-100, and J12.7-7-100 used a RT-2 prime coat between base and overlay slabs. Overlay slabs G12.14-100 and H12.7-7-100 were cast directly on the base slabs.
2. C stands for corner and T stands for transverse crack.
3. Coverages at failure are not indicated in report. The values in column 9 are rough numbers that might represent the failure coverages. They are obtained through the investigation of figures, deflection chart, and photographs.
4. C = 1.0 is used for the determination of equivalent rigid pavement design thickness.
5. Numbers in the parentheses represent the equivalent thickness for items H-12 and J-12 determined first by taking base 10", overlay 7", and determining the equivalent thickness (h_e'); then, using this (h_e') as existing base ($h_e' = h_e$) and overlay 7", determine the final design thickness. 20.30" and 18.30" thicknesses are determined using 7" overlay and 17" existing pcc concrete.

(Sheet 3 of 4)

Table 9 (Continued)

LOCKBOURNE TRAFFIC TEST NO. 3

37"

40,000 lb wheel load on twin-wheel load
Actual twin wheel load = [illegible] lb
Actual contact area = [illegible] sq. inch
Actual contract pressure = [illegible] psi

Item No.	Components of Pavement Thickness (inches): PCC Base Slab	Overlay Base	Binder and Wearing Course	Total Flexible Overlay Thickness (inches)	Total Thickness (inches)	Type of Overlay Base	"k" pci	"R" Average Flexural Strength of PCC Base (lb/in²)	Coverages at Failure	"F" Factor for A-type Traffic Area	Equivalent Rigid Pavement Design Thickness "h_d": $h_d = \frac{1}{F}(0.4t + Ch_e)$
1	6"	-	-	-	6	None	62	940	3	0.967	6.00"
2	6"	-	3	3	9	None	82	815	480	0.952	7.44"
3	6"	3	3	6	12	AC	68	815	-	0.962	8.73"
4	6"	6	3	9	15	Sand and Gravel	46	940	14	0.980	9.80"
5	6"	6	3	9	15	Sand Asphalt	58	827	500	0.970	9.90"
6	6"	6	3	9	15	Penetration Macadam	60	928	36	0.965	9.9[illegible]
7	6"	6	3	9	15	AC	81	839	7,500	0.953	10.0[illegible]
8	6"	6	3	9	15	Water Bound Macadam	47	940	-	0.978	9.60"
9	6"	9	3	12	18	Water Bound Macadam	51	773	-	0.97[illegible]	11.00"

1. C = 1.0 is used in the computations since the flexible pavement overlays are laid over the new 6-inch thick concrete base slab.
2. Coverages at failure are not indicated clearly in the report.

MAXWELL FIELD TESTS

37.5"

60,000 lb load on twin wheels
Tire pressure = 100 psi
(Contact pressure)

Item No.	Components of Pavement (inches): Aggregate Base Course	PCC Base Pavement	AC Overlay	"k" pci	"R" Average Flexural Strength of PCC Base (psi)	Coverage at Failure	"F" Factor for A-type Traffic Area	Equivalent Rigid Pavement Design[1] Thickness "h_d" (inches): $h_d = \frac{1}{F}(0.4t + Ch_e)$
A	12"	6"	6"	260	655	> 3,000	0.855	8.00
C	8"	6"	3"	260	614	> 3,000	0.855	6.67
B	12"	6"	6"	260	645	> 3,000	0.855	7.13

1. C = 0.75 is used in the computation since the existing PCC base pavement was two years old.

MACDILL FIELD TESTS

36"

60,000 lb load on dual wheel
1 coverage = 4 trips
Area = 267 sq. inch

36 inches and 267 sq. inches are assumed values, taking into consideration the figures.

Item No.	Components of Pavement (inches): PCC Base Pavement	Base Intermediate Layer	AC Overlay	PCC Overlay	Total Flexible Pavement Thickness "t" inches	"k" psi	"R" Average Flexural Strength of PCC Base (psi)	Coverages at Initial Cracking	Coverages at Failure	"F" Factor for A-type Traffic Area	Equivalent Rigid Pavement Design Thickness "h_d", inches: $h_d = \frac{1}{F}(0.4t + Ch_e)$	$h_d = \sqrt{h_r^2 + Ch_e^2}$	$h_d = h_r + 0.3\,Ch_e$
A	6	-	7	-	7	382	-	-	-	0.765	9.36		
1	6	-	3	-	3	436	774	-	-	0.740	7.70		
2	6	-	3	-	3	436	740	-	-	0.740	7.70		
3	6	-	5	-	5	401	799	-	-	0.740	8.78		
4	6	0.5" sand asphalt	-	6	-	401	883	198	965	-		6.97	6.63
5	6	-	7	-	7	401	800	-	-	0.740	9.86		
6	6	0.5" sand asphalt	-	8	-	401	820	356	2943	-		8.75	8.63
7	6	Lime Rock[6]	3		9	516	822	-	-	0.740	10.94		
8	6	Lime Rock[6]	3		9	558	-	-	-	0.740	10.94		

1. The "k" values are not given for each item separately. Thus the "k" value for the adjacent items are assumed to be the same.

2. Coverages at initial cracking and failure are determined by thorough investigation of plates (17-20) and using judgment. For sections covered by flexible overlay, no figures related to failure are submitted.

3. In this report static wheel load vs. deflection curves are given in detail.

4. C = 0.35 used, since the overlays are placed on a portion of an old runway (for PCC overlays).

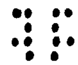
5. C = 0.75 used, since the overlays are placed on a portion of an old runway (for flexible overlays).

6. High "k" values are reached, since the subgrade soil was cohesionless.

(Sheet 3 of 4)

Table 9 (Concluded)

MULTIPLE-WHEEL HEAVY GEAR LOAD TESTS - SOUTH BOUND



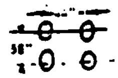
360 kips on 12-wheel assembly

1 traffic pattern = 10 coverages

Item No.	PCC Base Pavement "h_e" inches	AC Overlay "t" inches	"k" pci	"R" Average Flexural Strength (lb/in²)	Coverages at		"F" Factor for A-type Traffic Area	Equivalent Rigid Pavement Design Thickness "h_d"
					Initial Cracking	Failure		$h_d = \frac{1}{F}(0.4t + Ch_e)$
1	10"	-	140	620	192	392	-	10" (no overlay)
4	8"	-	125	620	48	240	-	8" (no overlay)
[illegible]	10"	4"	140	620	-	4,416	0.917	9.92"
[illegible]	8"	6"	125	620	-	4,416	0.325	9.08"

1. C = 0.75 is used in the calculation since the existing base was cracked due to the trafficking of a 12-wheel assembly.

MULTIPLE-WHEEL HEAVY GEAR LOAD TESTS - NORTH BOUND



240 kips on twin-tandem assembly

A = 267 sq. inches

1 traffic pattern = 10 coverages.

Item No.	PCC Base Pavement "h_e" inches	AC Overlay "t" inches	"k" pci	"R" Average Flexural Strength (lb/in²)	Coverages at		"F" Factor for A-type Traffic Area	Equivalent Rigid Pavement Design Thickness "h_d"
					Initial Cracking	Failure		$h_d = \frac{1}{F}(0.4t + Ch_e)$
1	10"	4"	140	485	40	680	0.917	9.94"
4	8"	6"	125	485	40	680	0.925	9.08"

1. C = 100 is used in the calculations since the existing base pavement was not cracked.

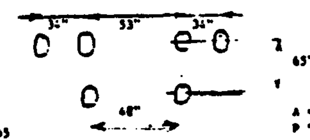
2. The contact tire pressures and contact tire areas are not given.

Load = 240 kip

Load = 60 kip

X = 34 Y = 65

X = 87 Y = 65



A = 285 sq. inch
p = 106 psi

(Sheet 4 of 4)

a. Items 21 and 22 in the Sharonville test--no coverages to failure were reported.

b. Fourteen items from the various tests in which traffic was discontinued before failure was obtained.

c. Items 1, 4, and 6 in Lockbourne test--an unusually small number of coverages to failure were obtained.

Computed maximum edge stress, design factor, flexural strength of concrete, gear load, gear configuration, and coverages to failure for each test item are summarized in Table 10. The following assumptions were made:

a. Thickness for the existing PCC, aggregate base courses, and AC courses were used when actual thicknesses were given to determine the "Equivalent Plain Rigid Pavement Design Thickness"; otherwise, design thicknesses were used for the items.

b. For items having bottom, middle, and top slabs, the thickness of the top slab was taken as the overlay thickness and the sum of thicknesses of the middle and bottom slabs was used as the existing slab thickness.

c. Maximum tensile edge stresses were determined using the modulus of subgrade reaction, k , corrected for saturation where such data were available; if not, k-values corrected for bending of the plate were used.

d. For flexible overlays, a C_b value equal to 1.00 was used for nontrafficked and newly constructed existing pavements, while a C_b value equal to 0.75 was used for trafficked and cracked existing pavements.

e. For rigid overlays, a C_r value equal to 1.00 was used for nontrafficked and newly constructed existing pavements, while a C_r value of 0.35 was used for trafficked and cracked existing pavements.

f. In the evaluation of flexible overlays, the flexural strength of the equivalent rigid pavement was assumed to be the same as the flexural strength of the existing PCC base pavement.

g. In the evaluation of rigid overlays, no correction factor was used since the differences between the flexural strengths of the existing PCC slabs and the rigid overlays were smaller than 100 psi for the items investigated.

h. For flexible overlays, an F-factor corresponding to "A" type traffic areas was used.

Using the data from the remaining 49 items, relationships between stress ratio and coverages, design factor and coverages, and rigid pavement thickness and coverages were examined. Using the least-squares technique, linear relationships between stress ratio, design factor, and rigid pavement thickness as dependent variables and the logarithm of coverages as the independent variable were determined.

Table 10

Pavement and Bending Information of Test Items Used in the Analyses

Traffic Test Section	Load and Gear Configuration	Item Number	Equivalent Thickness of Existing Pavement (inches)	Maximum Edge Stress "σ_e" (psi)	Flexural Strength of PCC Slab (R) psi	Design Factor $D.F. = \frac{R}{0.75\sigma_e}$	Stress Ratio $= \frac{\sigma_e}{R}$ (1)	Stress Ratio $= \frac{0.75\sigma_e}{R}$ (2)	"k" psi/inch	Coverages to Failure
Sharonville Traffic Test No. 1	100,000 lb; 37.5"; Contact Area = 267 in.[2]; TWIN-WHEELS	1	9.35	750	725	1.289	1.034	0.776	110	130
		2	12.28	516	725	1.873	0.712	0.534	105	950
		3	15.05	380	725	2.544	0.524	0.393	115	1,000*
		4	11.10	643	725	1.503	0.887	0.665	75	250
		5	10.02	742	725	1.303	1.023	0.970	75	200
		6	11.29	647	725	1.494	0.892	0.669	63	270
		7	11.86	586	725	1.650	0.808	0.606	75	1,250
		8	8.43	952	725	1.015	1.313	0.985	71	130
		9	9.56	764	725	1.265	1.054	0.790	90	485
		10	10.40	623	725	1.552	0.859	0.644	130	3,000*
		11	7.51	982	725	0.984	1.354	1.016	120	70
		12	8.81	829	725	1.116	1.143	0.896	100	1,150
		13	7.61	1,005	725	0.962	1.386	1.039	100	70
		14	6.76	1,110	725	0.871	1.531	1.148	125	60
		15	10.00	631	725	1.532	0.870	0.652	160	1,200
		16	8.8	847	725	1.141	1.168	0.876	160	1,600
Sharonville Traffic Test No. 2	100,000 lb; 37.5"; Contact Area = 267 in.[2]; TWIN-WHEELS	17	20.00	244	775	4.235	0.315	0.236	125	3,000*
		18	14.27	406	775	2.545	0.523	0.393	125	6,500*
		19	12.56	490	775	2.109	0.632	0.474	125	1,135
		20	12.43	496	775	2.083	0.640	0.480	25	1,000
		21	16.39	330	775	3.131	0.425	0.319	125	Not mentioned
		22	15.71	351	775	2.944	0.453	0.340	125	Not mentioned
		23	13.68	433	775	2.386	0.559	0.419	125	22,000*
		24	13.00	462	775	2.237	0.596	0.447	125	22,000*
		25	11.76	536	775	1.928	0.692	0.519	125	18,500
		26	10.61	618	775	1.672	0.797	0.598	125	1,200
		27	10.17	654	775	1.580	0.844	0.633	125	250
		28	8.43	835	775	1.128	1.077	0.808	125	230
		29	23.02	197	775	5.245	0.254	0.191	125	3,000*
		30	16.06	316	775	3.270	0.408	0.306	125	3,000*
		31	13.51	441	800	2.418	0.551	0.414	125	4,000
		32	10.92	594	800	1.796	0.743	0.557	125	1,200
		33	11.60	541	800	1.372	0.676	0.507	125	1,700
		34	9.19	746	675	1.206	1.105	0.829	125	90
		35	12.43	496	650	1.747	0.763	0.572	125	950
		36	10.30	637	650	1.361	0.980	0.735	125	215
		37	11.14	578	650	1.499	0.889	0.667	125	425
		38	9.00	757	750	1.321	1.009	0.757	125	170
		39	9.06	682	825	1.613	0.827	0.620	125	690
		40	11.46	555	825	1.982	0.673	0.505	125	4,620
		41	11.03	586	825	1.887	0.710	0.530	125	2,130
Sharonville Traffic Test No. 3	25,000 lb; Contact pressure = 200 psi	46	6.50	770	950	1.645	0.811	0.608	100	550
		47	6.18	816	950	1.552	0.859	0.644	125	400
		48	6.69	715	950	1.772	0.753	0.564	125	700
		49	7.30	620	950	2.043	0.653	0.489	125	1,600
		50	7.36	647	950	1.958	0.681	0.511	80	1,600
Sharonville Heavy Load Test Track	325,000 lb; 62.75"; A = 267 sq. in.; 31.25"	68	26.91	250	687	3.664	0.363	0.273	100	8,280
		69	22.67	304	692	3.035	0.439	0.329	100	4,040
		70	28.00	230	712	3.989	0.334	0.251	100	7,090
Lockbourne Field Tests No. 2	15,000 lb; Contact Area = 1,459 sq. in.	F 12 14-100	17.20	469	705	2.004	0.665	0.499	155	1,000
		G 12 14-100	19.80	377	840	2.971	0.449	0.337	160	1,430*
		H 12 7-7-100	20.30	385	760	2.563	0.520	0.390	106	1,725*
		J 12 7-7-100	18.30	402	785	2.604	0.512	0.384	215	370
		L 14 14-80	16.12	485	835	2.296	0.581	0.436	230	1,000
		M 14 14-80	16.12	506	785	2.069	0.645	0.483	175	887
Maxwell Field Tests	60,000 lb; 37.5"; Contact Pressure = 100 psi	1	8.00	443	655	1.971	0.676	0.507	260	3,000*
		C	6.67	549	614	1.491	0.894	0.671	260	3,000*
		B	7.13	509	645	1.690	0.789	0.592	200	3,000*
[illegible] Field Tests	60,000 lb; 36"; Contact Area = 267 sq. inch	4	6.97	275	853	4.136	0.322	0.242	401	983
		6	8.75	228	820	4.795	0.278	0.209	401	2,943
Lockbourne Traffic Test No. 3	60,630 lb; 37"; Contact Area = 668 sq. inch	1	6.00	854	940	1.469	0.909	0.681	62	3
		2	7.44	609	815	1.784	0.747	0.560	82	300
		4	9.80	470	940	2.622	0.500	0.381	46	14
		5	9.90	451	827	2.445	0.545	0.409	38	300
		6	9.91	447	920	2.760	0.482	0.361	50	26
		7	10.01	410	839	2.720	0.489	0.367	81	300*
Multiple Wheel Heavy Gear Load [illegible]	260,200 lb; A = 267 sq. inch; 30"; 44"	1	9.92	380	685	1.701	0.830	0.622	100	680
		4	9.98	406	685	1.687	0.841	0.631	125	680

Figure 12 illustrates the relationship between stress ratio and coverages. The resulting regression equation was

$$\text{Stress ratio, } \frac{\sigma_e}{R} = 1.83786 - 0.36218 \log \text{(coverages)} \qquad (13)$$

For this relationship, the standard error of the estimate was 0.190, the standard deviation was 0.276, and the correlation coefficient was 0.725.

The relationship for design factor versus coverages (Figure 13), was

$$\text{Design Factor, } \frac{R}{0.75\sigma_e} = 0.72327 \log \text{(coverages)} - 0.26397 \qquad (14)$$

For this expression, the standard error of the estimate was 0.595, standard deviation was 0.702, and the correlation coefficient was 0.529. In carrying out the regression analysis for Equation 14, items 68, 69, and 70 in the Sharonville Heavy Load test track and item J12-7-7-100 in the Lockbourne test were not considered because their results deviated considerably from the others.

The relationship between equivalent rigid pavement thickness, h_d, and coverages (Figure 14), was

$$h_d = 1.825073 \log \text{(coverages)} + 5.018118 \qquad (15)$$

For this expression, the standard error of the estimate was 2.334, the standard deviation was 2.515, and the correlation coefficient was 0.372. As with Equation 14, Items 68, 69, and 70 in the Sharonville Heavy Load test track and Item J12-7-7-100 in the Lockbourne test were not included in the regression analysis.

From an examination of Figure 13, Monismith, Yücë, and Finn[5] observed that almost 90 percent of the items have yielded rather small numbers of coverages for the corresponding design factors. When the regression equation was compared with current Corps of Engineers' criteria, it was noted that a steeper slope was obtained, suggesting, in turn, that the current criteria might be "unconservative." Accordingly, an analysis was made to ascertain the probable final form of Equations 11 and 12 for bituminous and concrete overlays which might fit the current design criteria.

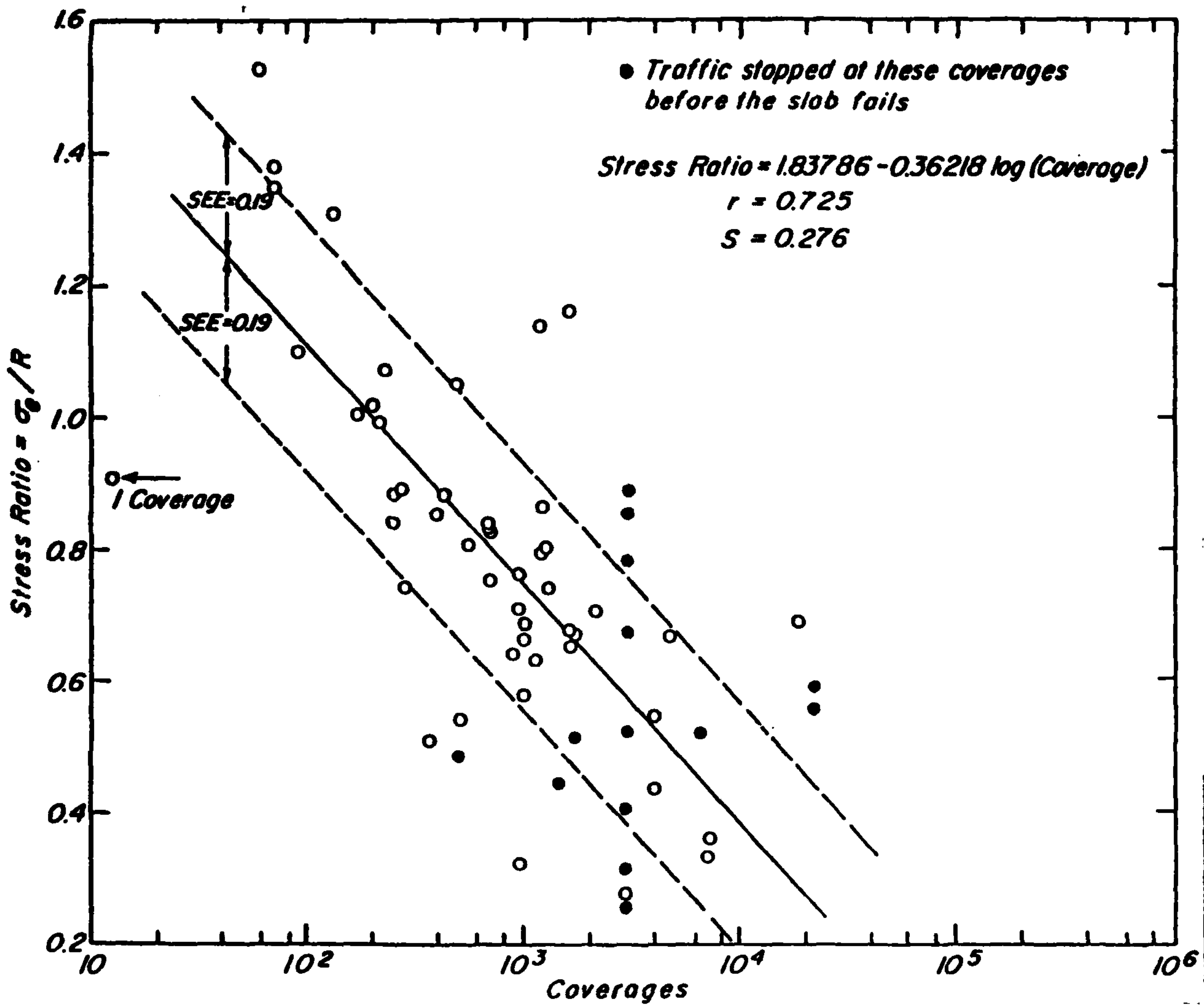


Figure 12. Relationship between stress ratio and coverages to failure for rigid and flexible overlays (from Monismith, Yücë, and Finn[5])

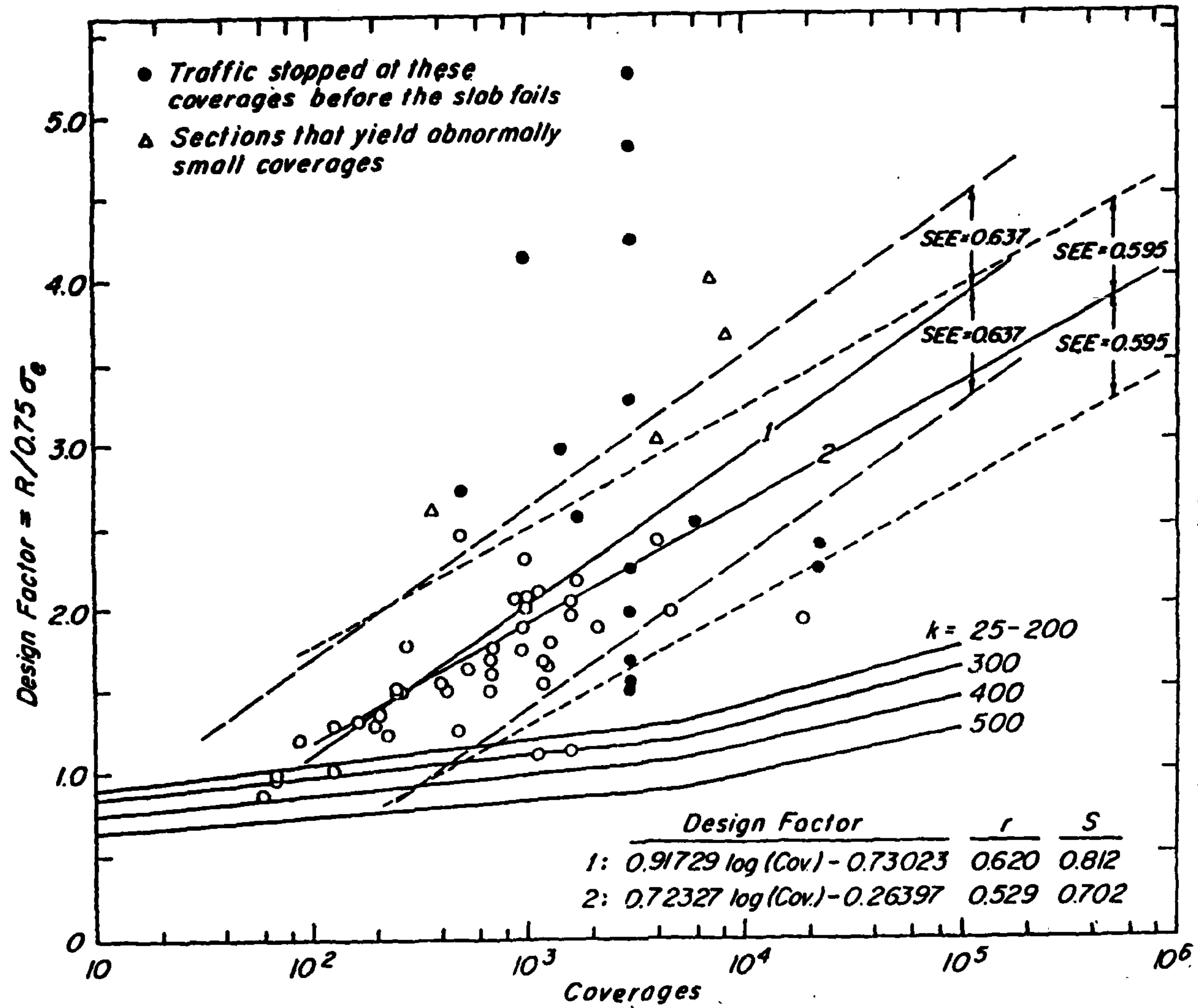


Figure 13. Relationship between design factor and coverages to failure for rigid and flexible overlays (from Monismith, Yüce, and Finn[5])

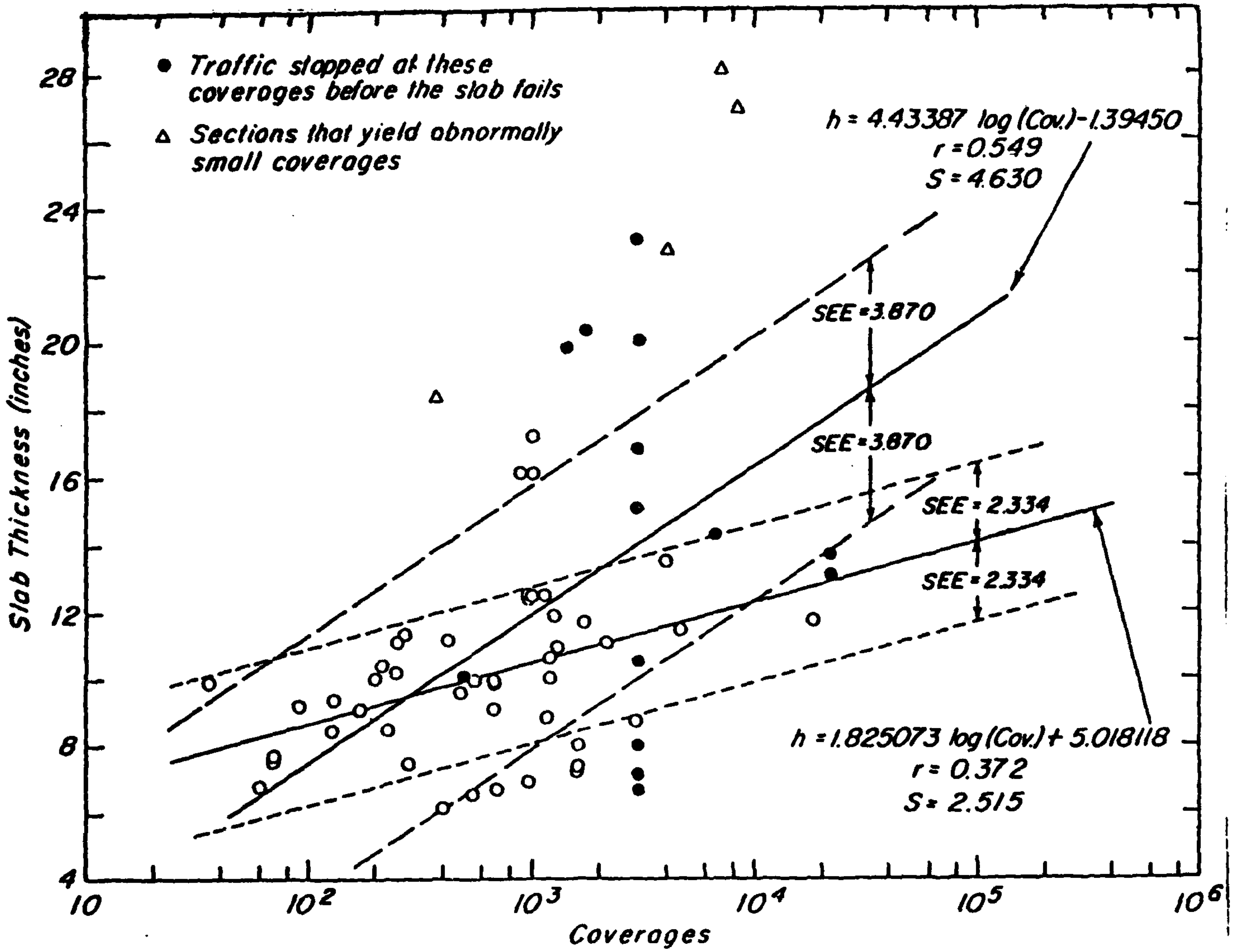


Figure 14. Relationship between slab thickness and coverages for rigid and flexible overlays (from Monismith, Yücë, and Finn[5])

Bituminous and concrete overlays were then examined separately. Monismith, Yücë, and Finn[5] commented that while the data were not sufficient to establish firm design relationships, guidelines were provided for further investigation. In the analysis for bituminous overlays, the data from 30 items were used. On the other hand, the data from only 12 items were available for concrete overlays. Of these 12 items, 9 contained nonbonded concrete overlays, 2 contained partially bonded overlays, and 1 contained a bonded concrete overlay. Data from the 9 unbonded items were used.

Bituminous Overlays. For this analysis, Equation 11 was modified to permit different coefficients (equivalencies) for the bituminous-bound and untreated aggregate layers to be considered. Equivalencies obtained from Reference 11 are:

1.7-2.3 (average 2.0) - for bituminous surface course

1.4-2.0 (average 1.7) - for crushed aggregate base course

The resulting equation had the following general format:

$$h_d = \frac{1}{F}(At_1 + 0.85(At_2) + Ch_e) \tag{16}$$

where

h_d , F , C , and h_e have been defined previously in Equations 11 and 12

A = overlay thickness coefficient

t_1 = thickness of bituminous bound layers, in.

t_2 = thickness of unbound layers, in.

The coefficient 0.85 was obtained from the ratio of the average equivalencies for the unbound and bound layers, i.e., 1.7/2.0.

With values for the coefficient A of 0.40, 0.35, 0.30, and 0.25 and values for C of 0.75, 0.60, 0.50, and 0.35, corresponding thicknesses, maximum tensile edge stresses, and design factors for each item were calculated.

The design factors were compared with the performance criteria of the Corps of Engineers in Figures 15 and 16. From an examination of these figures, it appeared that the following equations provide a better comparison with the criteria than Equation 11:

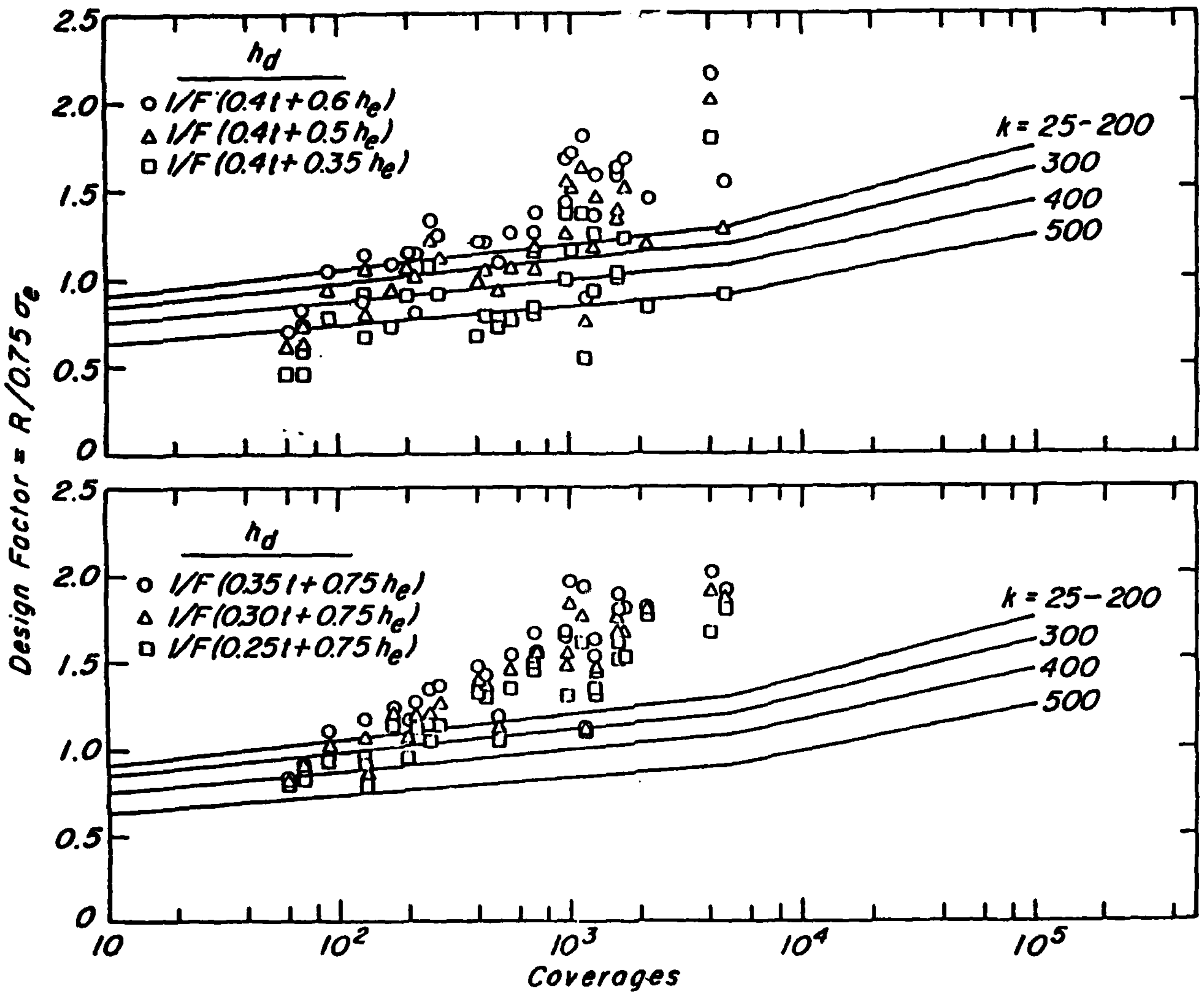


Figure 15. Comparison of modified design expressions with current Corps of Engineers limiting stress performance criteria, flexible overlay (from Monismith, Yücë, and Finn[5])

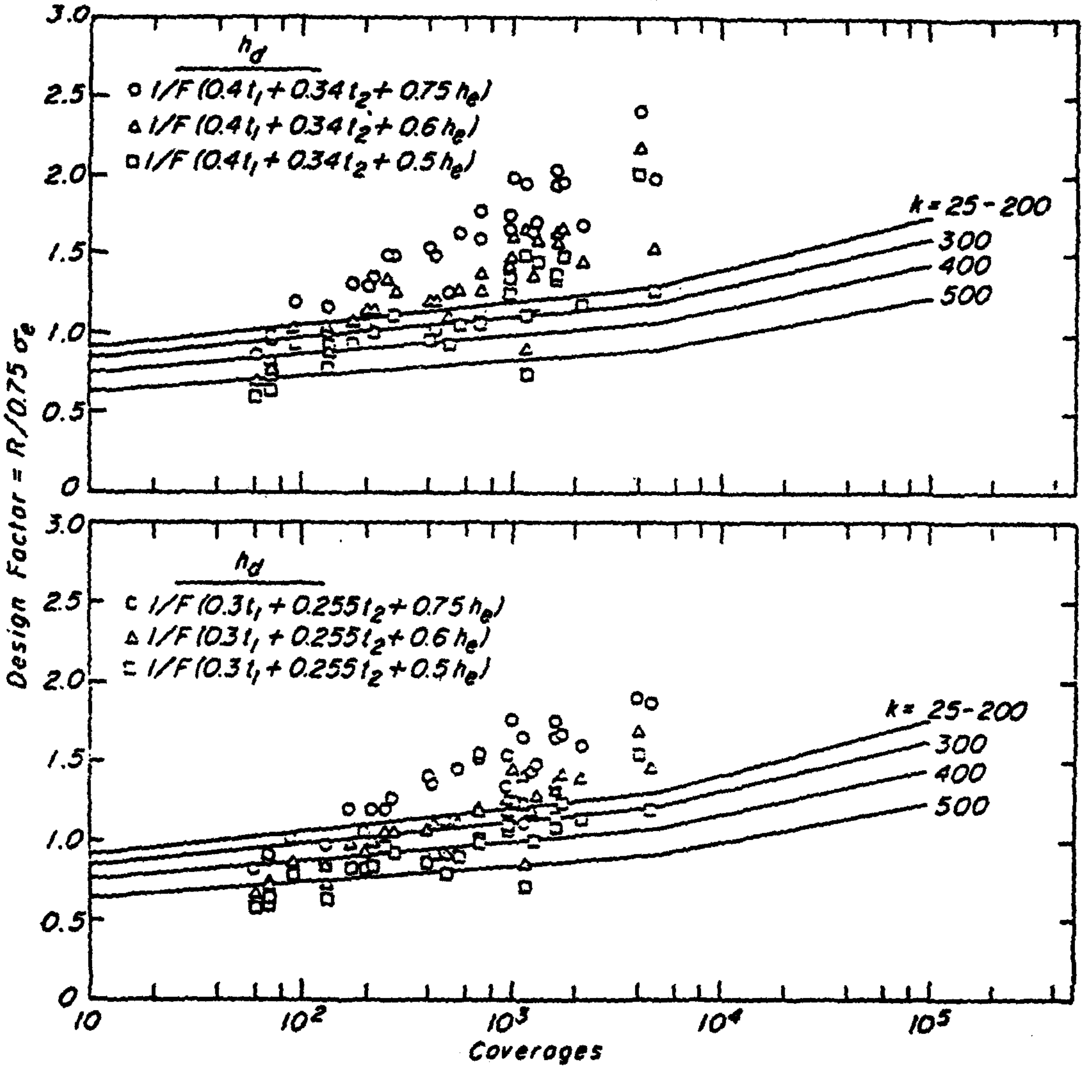


Figure 16. Comparison of modified design expressions with current Corps of Engineers limiting stress performance criteria, flexible overlay (from Monismith, Yücë, and Finn[5])

$$h_d = \frac{1}{F} (0.4t_1 + 0.34t_2 + 0.5h_e) \quad (17)$$

or

$$h_d = \frac{1}{F} (0.3t_1 + 0.255t_2 + 0.6h_e) \quad (18)$$

Concrete Overlays. Because of the very limited amount of data, only the nonbonded overlay case was examined. As with the bituminous overlay case, the design relation (Equation 12) was modified, in this instance, as follows:

$$h_d^x = Ah_o^2 + Ch_e^2 \quad (19)$$

where

h_d, h_o, h_e, and C are as already defined

A = overlay thickness coefficient

x = coefficient for equivalent rigid pavement design thickness

Because the field data were limited (only nine items as noted earlier), the analysis was confined to values for A of 0.9, 0.8, and 0.7 and 2.1, 2.2, and 2.3 for the exponent x. For newly constructed and untrafficked pavements, C was taken as 1.0, while for the failed and heavily trafficked sections, a value for $C = 0.35$ was used.

Equivalent rigid thicknesses and the corresponding maximum tensile edge stresses were determined for the range in values of A and x. Design factors were then determined and are plotted in Figure 17. If the values for the Sharonville Heavy Load test track (items 68 and 69, coverages 8280 and 4240, respectively) were excluded as was done earlier, the following equation appeared to best fit the current design criteria:

$$h_d^{2.3} = h_o^{2.0} + Ch_e^{2.0} \quad (20)$$

Based on the results of reanalyses of the Corps of Engineers' traffic data, Monismith, Yücë, and Finn[5] provided the following conclusions:

a. Ninety percent of the items from ten different traffic tests yielded fewer coverages to failure than would be indicated by current Corps of Engineers' criteria. Thus it would appear that the current criteria may be unconservative when applied in this manner to overlay design.

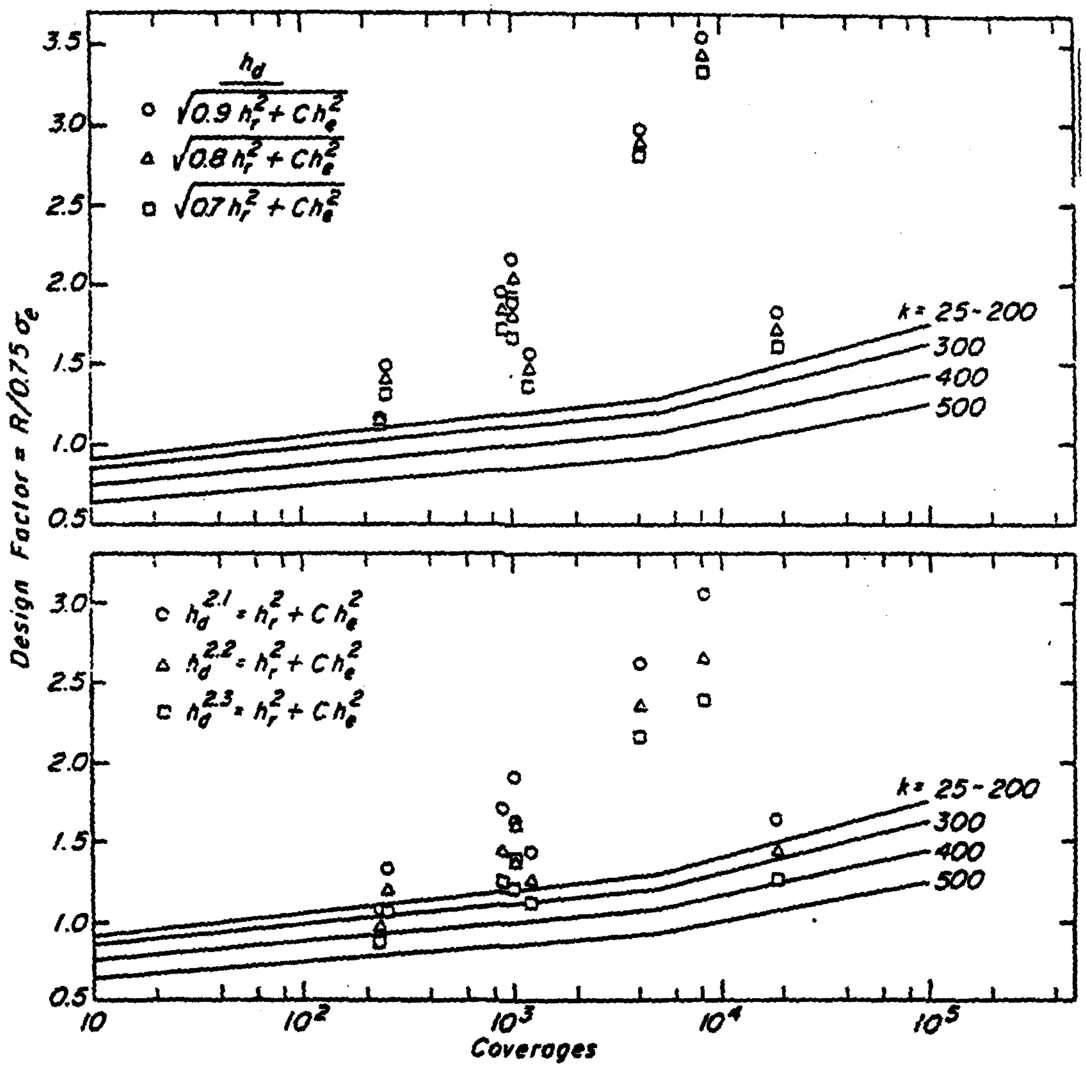


Figure 17. Comparison of modified design expressions with current Corps of Engineers limiting stress performance criteria, rigid overlay (from Monismith Yücë, and Finn[5])

b. The relationship for design factor versus coverages yielded a steeper slope than those developed by the Corps for a range in k-values.

c. The relationship between stress ratio and coverages appeared to correlate with existing data to a better degree than either the relationship between design factor and coverages or slab thickness and coverages.

d. In the evaluation of flexible overlays, the C value for failed sections should not be limited to 0.75. As with rigid overlays, a range from 1.00 to 0.35 may be appropriate. This analysis indicated that a value of 0.5 or 0.6 for C would provide a closer correlation with the existing criteria than a value of $C = 0.75$.

e. For flexible overlays incorporating unbound base courses and bituminous-bound surface courses, it would appear reasonable to apply different coefficients to the treated and untreated layers as illustrated in Equation 17 or 18. Equivalency factors listed in Reference 11 can serve as a basis for developing these coefficients.

Monismith, Yücë, and Finn[5] stated that improvements in the current FAA overlay design methodology for rigid pavements can be warranted if modified design equations represented by Equations 17 and 18 can be used. In conjunction with this step, field trials should be conducted to permit validation of the suggested improvements.

A comparison was made among the modified equations for nonbonded concrete overlays proposed by Monismith, Yücë, and Finn[5]; Chou[9]; and Lytton.[8] The equations are

FAA design equation	$h_d^2 = h_o^2 + Ch_e^2$	(4)
Monismith, Yücë, and Finn[5]	$h_d^{2.3} = h_o^2 + Ch_e^2$	(20)
Chou[9]	$h_d^{2.5} = h_o^{2.5} + Ch_e^{2.5}$	(21)
Lytton[8]	$h_d^3 = h_o^3 + Ch_e^3$	(22)

The variables are all as previously defined.

By assuming $C = 1$ and the thicknesses of rigid overlay h_o were calculated for various h_e and h_d , the results are presented in Table 11. As seen in the table, the thicknesses of concrete overlays computed by the expressions proposed by Monismith, Yücë, and Finn;[5] Chou;[9] and Lytton[8] are all larger than those computed by the existing FAA design equation, with the greatest differences between those computed by the existing equation and by Monismith, Yücë, and Finn. It is particularly true for thicker existing concrete pavement h_e .

Table 11

Thicknesses of Concrete Overlays, h_o

Existing Concrete Pavement, h_e in.	New Pavement h_d in.	FAA Current Design Equation in.	Monismith, Yücë, and Finn in.	Chou in.	Lytton in.
4	6.0	4.5	6.8	5.0	5.3
4	8.0	6.9	10.2	7.4	7.7
4	10.0	9.2	13.5	9.6	9.8
8	10.0	6.0	11.6	7.1	7.9
8	12.5	9.6	16.4	10.7	11.3
8	15.0	12.7	21.1	13.7	14.2
12	13.0	5.0	14.9	6.6	7.8
12	15.0	9.0	19.1	10.7	11.8
12	18.0	13.4	25.0	15.0	16.0

Anomalies of the Design Procedures. Chou[9] suggested that a section be added to the FAA design manual for the consideration of composite pavements and other areas where design results could conflict with one another when different procedures are used.

NEW OVERLAY DESIGN PROCEDURES

Both Monismith, Yücë, and Finn[5] and Darter and Smith[6] suggested immediate improvements of the FAA existing design methodology and the development of completely new procedures using the best available information. Both references state that the existing FAA overlay design procedure is based on the thickness deficiency approach which has serious problems, and the new procedure should be analytically based, i.e., the use of mechanistic model to calculate stress, deflection, fatigue damage, etc. (It should be pointed out that the new structural design procedure for rigid pavements developed at the WES[17] is mechanistic in nature.) In the use of the mechanistic approach, both references stated strong preferences for the finite element approach over other approaches.

Monismith, Yücë, and Finn noted that there are a number of analytical tools now available, particularly the finite element procedure, which can be used in the development of improved design methodology so that more alternatives can be considered.[5] Coetzee and Monismith showed an example of the use of such procedures to examine the performance of interlayers to mitigate reflection cracking.[35] The improved methodology based on such techniques would thus have more flexibility to consider new materials and their combinations as they might be developed for overlay design.

Monismith, Yücë, and Finn suggested that the analytically based procedure for new overlay design should utilize the results of research which have been analyzed in recent years.[5] An example of the type of procedure which might be considered is that developed by Austin Research Engineers (ARE) for the Federal Highway Administration (FHWA).[36] This procedure, the framework for which is shown in Figure 18, considers both rigid and flexible overlays on rigid pavements. For the PCC (rigid) overlays jointed concrete pavement (JCP) or continuously reinforced concrete pavement (CRCP) are possible alternatives.

The approach recommended by Darter and Smith[6] for developing new overlay design procedures to provide "equivalent" design requires the consideration of both mechanistic concepts and the PCI performance evaluation. The use of

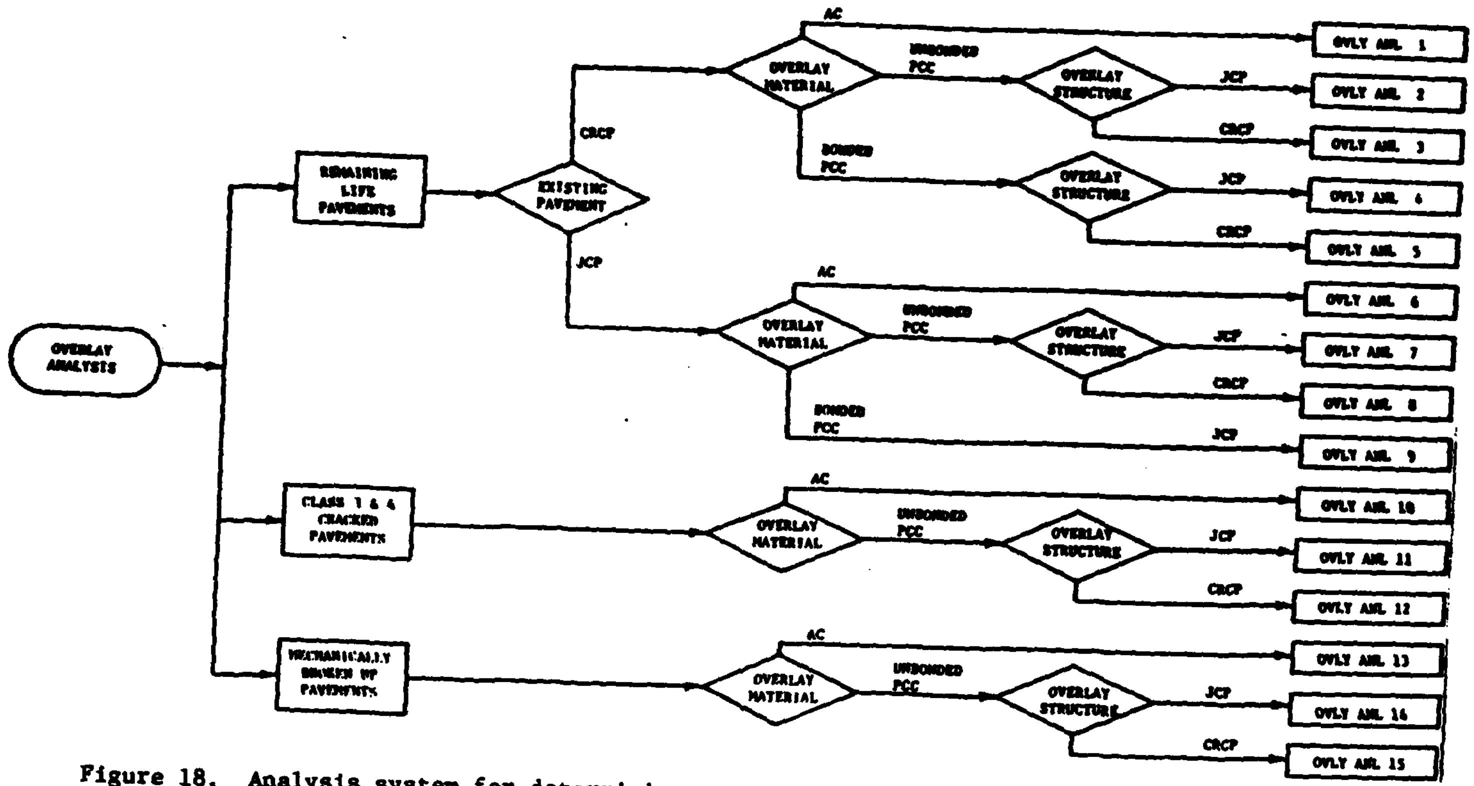


Figure 18. Analysis system for determining overlay thickness for existing rigid pavements considering wheel load repetitions (from Austin Research Engineers[36])

finite element techniques permits the accurate structural modeling of overlays and joint systems. The PCI concept, as they claimed, is the only known procedure for determining "equivalent" performance of bituminous and various concrete overlays.

Darter and Smith[6] strongly urge the use of finite element technique in the development of new overlay design procedure. They claim that most structural failures in concrete pavements occur or start at a joint. To model a joint either finite element analysis or Westergaard equations must be used. However, Westergaard equations can model only a free edge or free corner. Joint load transfer plays a dramatic role in edge and corner stress. To effectively model a joint or corner with load transfer, finite element systems must be used. Darter and Smith[6] suggested the finite element programs ILLISLAB[30] available at the University of Illinois and WESLIQID and WESLAYER[30] available at WES.

Darter and Smith[6] also suggested a way to improve the overlay design equations to account for the difference in layer properties by a rational method. It is relatively simple to measure the in-place flexural strength and to estimate the overlay flexural strength in design. A more logical method would be to run a series of analyses to develop a relationship. This could then be put in graphical or tabular form for ease of use by a design engineer. Figure 19 shows the effect of modulus of elasticity of overlay on critical stress in a base slab. A factor so developed could be applied to the existing thickness for either increasing or decreasing the thickness as needed. Figure 20 shows how the critical tensile stress in the base slab decreases with increasing bituminous overlay thickness.

Darter and Smith[6] emphasized the advantage of using the finite element technique to calculate the abrupt increase in stresses in the overlay concrete layer when a crack exists or develops in the base slab, for the crack will propagate through the bonded or partially bonded concrete overlay. Darter and Smith[6] used the ILLISLAB finite element program to compute the stress in the overlay when a crack with no load transfer is below the loaded area. Table 12 presents the results of that analysis. These results show the very high stress in a bonded concrete overlay with a crack in the base slab and show why the crack will quickly propagate to the surface. In the bituminous overlay, the stress is tensile above the crack, which would likely lead to reflection through the overlay.

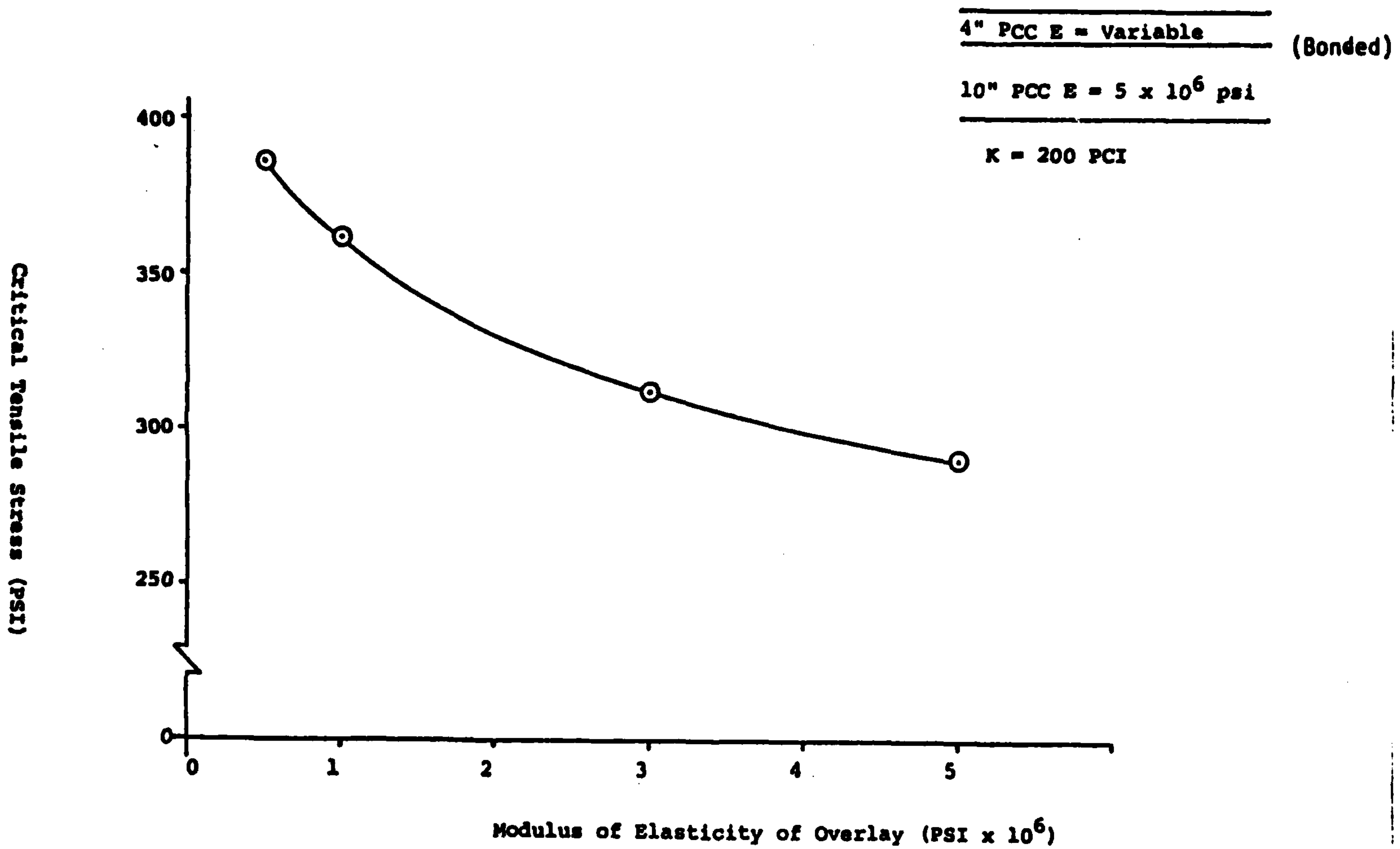


Figure 19. Effect of modulus of elasticity of overlay on critical stress in base slab (from Darter and Smith[6])

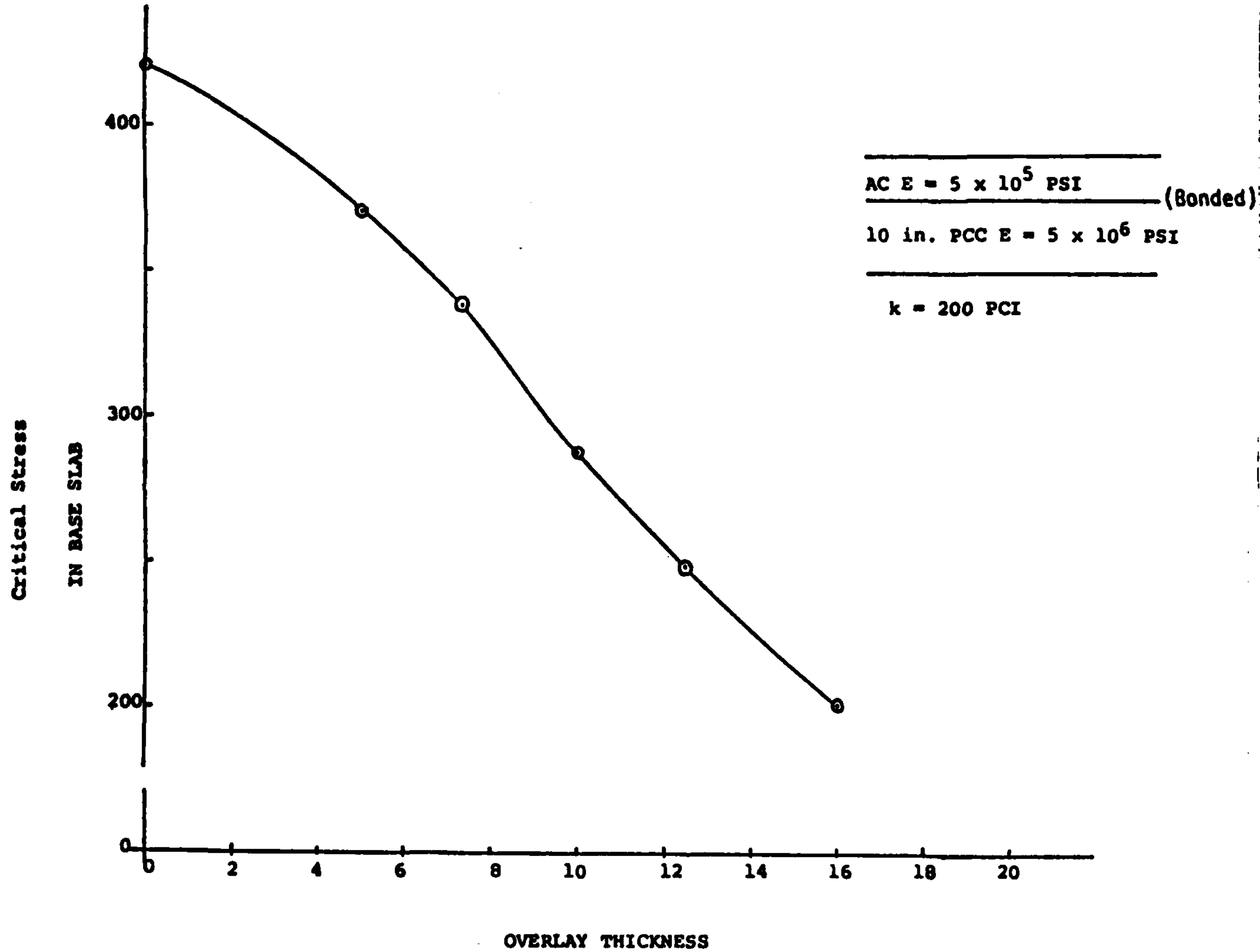


Figure 20. Effect of asphalt overlay thickness on base slab critical stress (from Darter and Smith[6])

Table 12

Critical Stress with a Crack in the Base Slab

Overlay Type	Overlay Thickness in.	Bond Type	No Crack in Base: Base Stress psi	No Crack in Base: Overlay Stress psi	With Cracked Base: Base Stress psi	With Cracked Base: Overlay Stress psi
PCC	4	Bond	290	--	--	490
PCC	9.8	Unbonded	267	261	--	398
AC	10	Bonded	288	--	--	104

Darter and Smith[6] reiterated that the purpose of using a finite element model is to more accurately characterize the overlay and existing structure. By knowing the stress in the base slab and overlay and knowing the properties of the overlay, the thickness of the overlay could be designed to limit the stress (and resulting fatigue damage) in the base slab and overlay to a level which would provide an acceptable amount of cracking in the base slab and propagation through the overlay. Figure 21 illustrates the basic concept.

LONG-RANGE PROGRAMS

The Board of Investigators convened at WES on 9-10 Feb 1982 for the purpose of discussing overlays for rigid pavements. At the completion of the conference, the Board of Investigators prepared a combined report to the FAA based on the recommendations of the Board. The report recommended both short-term improvements and long-range research programs. The long-range programs include (a) use of the mechanistic approach, (b) long-term pavement monitoring, (c) use of nondestructive testing, (d) use of probabilistic concepts in overlay design, (e) surface drainage, and (f) predicting maintenance and rehabilitation activities. The details of the long-range program can be found in Appendix B, the report of the Board of Investigators.

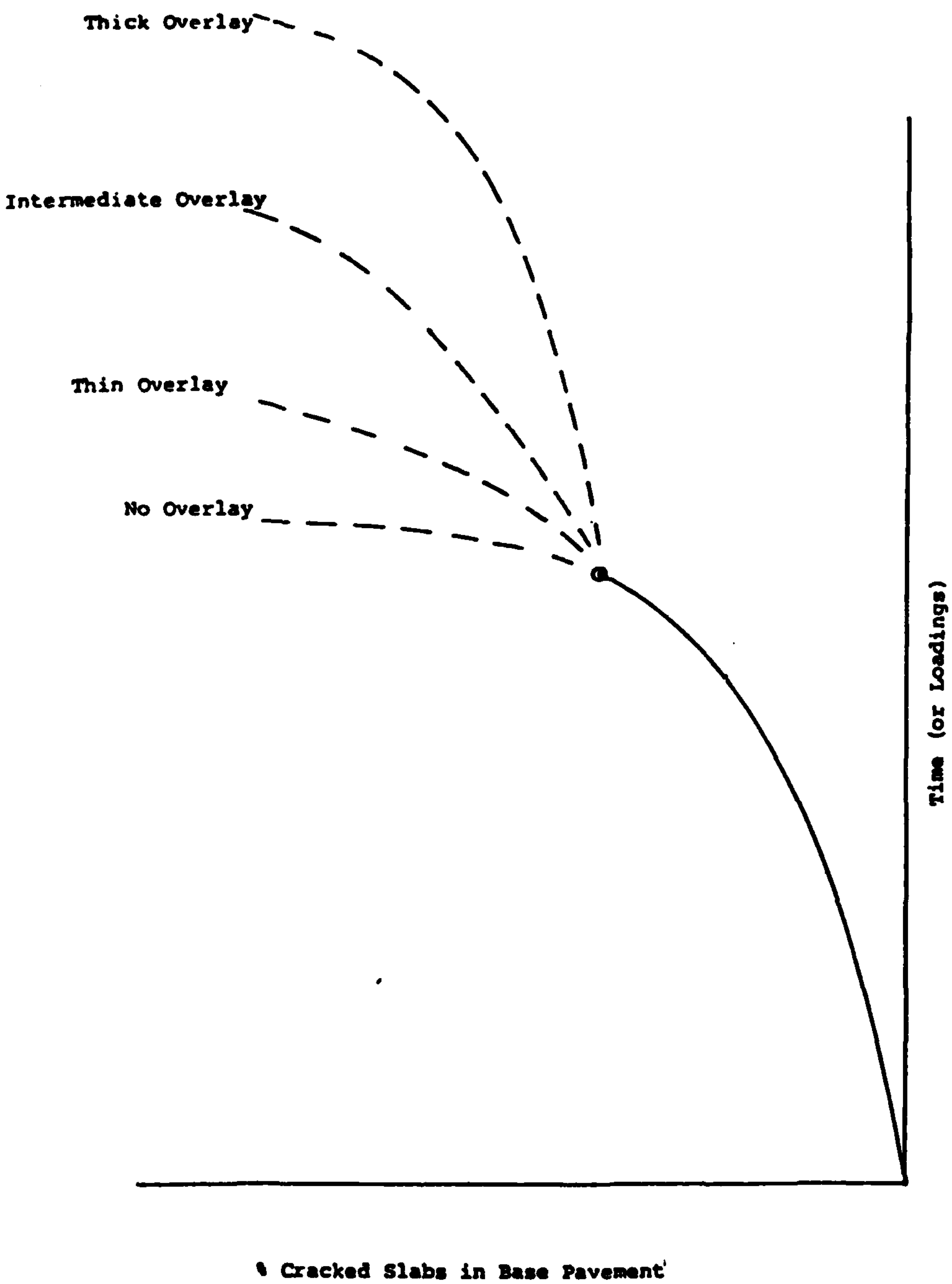


Figure 21. Effect of overlay thicknesses on the rate of slab cracking in base slab

IMMEDIATE IMPROVEMENTS OF FAA OVERLAY DESIGN PROCEDURES

The existing FAA overlay design procedures have been shown to have deficiencies. Immediate improvements can be made and suggested corrections and modifications are presented in this chapter in the following order:

- a. Uniform procedures to determine the design flexural strength of the PCC and k-values.
- b. Use of nondestructive testing (NDT) to determine foundation k-values.
- c. Use of NDT to detect voids under the slab.
- d. Measurement of load transfer.
- e. Improved condition survey methods.
- f. Definition of failure.
- g. Verification and modification of design equations.
- h. Anomalies of the design procedures.

UNIFORM PROCEDURES TO DETERMINE THE "DESIGN" FLEXURAL STRENGTH OF THE PCC AND k-VALUE

Guidelines for the selection of design flexural strength of the PCC and k-values should be provided in the FAA Advisory Circular,[11] Chapter 3, Section 3, paragraphs 44a and 44b and in Chapter 4, paragraph 64. The suggested revisions are between the asterisks (*) as follows.

44. DETERMINATION OF CONCRETE SLAB THICKNESS.

a. Concrete Flexural Strength. The required thickness of concrete pavement is related to the strength of the concrete used in the pavement. Concrete strength is assessed by the flexural strength method as the primary action of a concrete pavement slab is flexure. Concrete flexural strength should be determined by ASTM C 78 test method. Normally a 90-day flexural strength is used for design. Item P-501 of AC 150/5370-10 is specified in terms of the 28-day flexural strength of the concrete. This is done to reduce the curing of the specimens to a practical time. The designer can safely assume the 90-day flexural strength of concrete will be 10% higher than the 28-day strength except when high early strength cement or pozzolanic admixtures are used. When either high early strength cement or pozzolanic admixtures are used, the 28-day flexural strength should be used for design. *The design concrete strength can be determined from the equation

$$DFS = MFS - 1.0SD$$

where

DFS = design flexural strength, psi

MFS = mean flexural strength from test data, psi

SD = standard deviation of flexural strength, psi

This provides for approximately 85 percent level of confidence for concrete strength. A sufficient number of strength tests should be conducted to determine the mean and the standard deviation of the flexural strength.*

b. k-Value. The k-value is, in effect, a spring constant for the material supporting the rigid pavement and is indicative of the bearing value of the supporting material. *The same equation used to determine the design concrete flexural strength presented in paragraph 44a can be used to determine the design k-value.*

64. MATERIAL SELECTION CONSIDERATIONS. Criteria are presented in this chapter for both bituminous and concrete overlay pavements. The selection of the overlay type should be made after careful consideration of many factors. The designer should consider the total life cycle cost of the overlay pavement. Life cycle costs should include initial construction and maintenance costs over the design life of the pavement. Other considerations such as allowable down time of the pavement and availability of alternate pavements to use during construction will have a significant impact on the overlay type selected. *For the design of overlays for concrete pavements, the concrete flexural strength should be properly selected based on the following criterion

Type of Overlay	Design Flexural Strength
Bituminous concrete	Existing concrete slab
Fully bonded concrete	Existing concrete slab
Partially bonded concrete	Overlay concrete slab
Unbonded concrete	Overlay concrete slab

This criterion results in logical selection of the strength for the layer where stress is more critical. The actual value of flexural strength of the existng slab should be determined from either (a) coring the slab at several locations (20 cores minimum) and then testing for indirect tensile strength (and then transposing this strength into flexural strength), or (b) cutting standard sized beams from a few slabs and directly measuring their flexural

strength. Once the mean and standard deviation of the flexural strengths are computed, the design flexural strength can be determined from the equation shown in paragraph 44a.*

USE OF NONDESTRUCTIVE TEST (NDT)
TO DETERMINE FOUNDATION k-VALUES

It is recommended that a new subparagraph be added to the existing FAA design manual as follows.

43. DETERMINATION OF FOUNDATION MODULUS (k-VALUE) FOR RIGID PAVEMENT.

*d. Determination of k-value from the Nondestructive Testing (NDT) Procedure. The conventional procedure to determine the k-value using the plate-bearing test is difficult, expensive, and time consuming, and thus is only performed at a few locations on a pavement facility. NDT can be used to determine an elastic repeated load k-value at the center of a slab by back-calculation through standard Westergaard-type equations or influence charts. The repeated load k-value is then reduced to the standard gross k-value through an empirical factor. This conversion is not needed if a static load is used. It is to be noted that the tests should be conducted using a heavy load and at times when the temperature gradient is approximately zero through the slab; otherwise, a temperature correction should be made. This procedure to determine k-value using NDT can reduce cost and time and actually provide more reliable foundation support data. Many more tests could be conducted and the variation of slab support over the pavement feature can be determined. When subbase or stabilized subbase is used under the slab, the k-value determined by NDT is not the subgrade k-value, but the composite value of the entire foundation support. The relationship between the deflection at the center of a concrete slab and the subgrade k-value can be computed by the following formulas:

a. For positive temperature differentials (nighttime condition)

$$k = \exp\Big(-0.531 - 2.058 \log_e h + 6.078/h - 1.948 \log_e w + w\,(1.36\, h + 0.354\, h^2) - 10.16\, h^2 w^2 - t^2\,(0.748 - 0.0011\, h - 16\, w + 0.133 \log_e w + 219.2\, w^2)\Big)$$

b. For negative temperature differentials (daytime condition)

$$k = \exp\Big(7.739 - 38.99\, wh + h^2\,(0.00015 \log_e w + 101.9\, w^2) - t^2\,(7.76\, w - 0.0277\, h - 486.5\, w^2 + 11.9\, h^2 w^2)\Big)$$

The multiplying factor for correcting the k-value for the magnitude of the load, that is different from 14,000 lb, is

$$\left(\frac{P}{14,000}\right)^{\exp(2.3A)}$$

and the correction factor for deflection w_1 is shown below. The factor is to correct the errors caused by conditions where the concrete modulus E and the slab size are, respectively, different from 4,000,000 psi and 15 ft:

$$w = \exp\left(\frac{1}{C}\left(-0.0071\ (L-15) + 6.4\left(\frac{1}{L} - \frac{1}{15}\right) - (\log_e L - \log_e 15)\ (0.1738\ h + 0.288\ \log_e h) + D\ \log_e\left[w_{LE}\left(\frac{E}{4000000}\right)^{\exp(B/A)}\right]\right)\right)$$

where

- w = deflection, inches. The deflections are to be corrected for E and L values other than 4,000,000 psi and 15 ft
- w_{LE} = the deflections associated with given values of slab size L and concrete modulus E
- P = the magnitude of the load, lb
- h = slab thickness, inches
- L = slab size, ft
- E = Young's modulus of the concrete slab
- t = temperature differential, Fahrenheit degrees per inch of concrete
- $A = 0.29417 - 0.00733\ h$
- $B = -0.0297 - 0.01474\ h$
- $C = -2.621 + 0.506\ \log_e h - 0.0138\ h$
- $D = -2.064 + 0.506\ \log_e h - L\ (0.049 + 0.00092\ h) + 0.00079\ L^2$
- $\exp(A) = e^A$

The equations are derived based on computations that the single load (14,000 lb) is placed at the slab center, and the diameter of the circular load can vary from 6 to 24 in. The thickness of the concrete slab ranges from 6 to 20 in., and the size of the concrete slab ranges from 10 to 25 ft and the slab should be nearly square in shape. The finite element computer program was used to establish the equations and with the assumption that the subgrade support is uniform.*

The derivation of the equations is rather tedious and is presented in Appendix D.

USE OF NDT TO DETECT
VOIDS UNDER THE SLAB

It is recommended that a new paragraph be added to Chapter 4 after paragraph 75 in the existing FAA Advisory Circular as follows.

"76. DETECTION OF VOIDS UNDER SLABS. Voids may develop beneath joints and corners of old concrete slabs to be overlaid. The existence of such voids would seriously shorten the service life of the overlay pavements. It is suggested where voids may be suspected that heavy-load NDT be used to determine and locate any existing voids. All slabs with voids should be undersealed before any overlay is placed. The finite element computer program which is capable of computing deflections at the joint and corners and other locations of the concrete slabs or other techniques may be used to assist the determination and location of the voids."

MEASUREMENT OF LOAD TRANSFER

Work has been done at the WES to establish a relationship between load transfer (or stress transfer) and deflection transfer (or deflection load transfer or shear transfer) across a joint in a concrete pavement. The load transfer is taken to be the ratio of the flexural stress in the concrete on the unloaded side of the joint (σ_u) to that of the total stress (the sum of the stresses on both loaded and unloaded sides ($\sigma_u + \sigma_L$)) in percent. The deflection transfer is computed as the ratio of vertical deflections along the joint between the unloaded (W_u) and loaded (W_L) slabs in percent. The term "load transfer" has been used for years by the U. S. Army Corps of Engineers. The term "shear transfer" was first used by the Bureau of Public Roads (now the Federal Highway Administration) to define the deflection transfer across a joint. The Corps of Engineers as well as the FAA assumes 25 percent load transfer across a joint in good condition. Because load and stress are proportional, this means the stresses at a joint can be reduced from the free edge stress by 25 percent where good load transfer can be expected.

The finite element computer program WESLIQID was used to establish a relationship between the measured deflections at a joint and the load transfer

capability of the joint. The layout of the NDT on a pavement surface is shown in Figure 22 for the WES 16-kip vibrator and the falling weight deflectometer (FWD). The applied load in both cases is equal to 14,000 lb. Stresses and deflections were computed near the load for pavements with various slab thickness, h , concrete modulus, E , and subgrade k-value. The dimensions of the concrete slabs were assumed to be 20 by 20 ft. To establish such a relationship, various sizes of dowel bars spaced at 12 in. apart were assumed along the joint for a given pavement condition and the computation was made. The deflection ratio (or shear transfer (W_u/W_L)) between points A' and A (see Figure 22) and the load transfer $(\sigma_u/(\sigma_u + \sigma_L))$ at points B' and B for a given pavement (and for a particular dowel bar size) are plotted as a data point in Figure 23. Percent maximum edge stresses, defined as the ratio of the stress at point B to the maximum free edge stress in percent, are also plotted in the figure. Table 13 contains the properties of the pavements used in the computations.

The percent maximum edge stress plotted in Figure 23 is the ratio in percent of the stress at point B to the maximum free edge stress at point B due to the load at A. The latter is always at point B. It should be noted that the maximum stress in a jointed pavement, when the load is placed close to the joint as shown in Figure 22, is not always at point B. When the two slabs are connected by good load transfer devices, the two slabs act as a single slab and the (edge) load approaches an interior load and thus the maximum stress occurs at the load center. On the other hand, the location of the maximum stress moves to the slab edge (point B) when the load transfer capability across the joint becomes very poor. When load transfer at the joint is zero, the stress at point B is the maximum edge stress or maximum free edge stress.

Figure 23 shows that when deflections at the plate center (point A) and at the location of the detector (point A') are measured, the percent load transfer or the percent maximum edge stress can be determined. When load transfer is considerably less than 25 percent (or the maximum edge stress is considerably greater than 75 percent), mechanical devices should be installed to improve load transfer at the joint. Otherwise reflective cracks will occur in the overlays.

Figure 23 also shows that the relationships are not very sensitive to pavement thickness, concrete modulus, and subgrade support. Although there

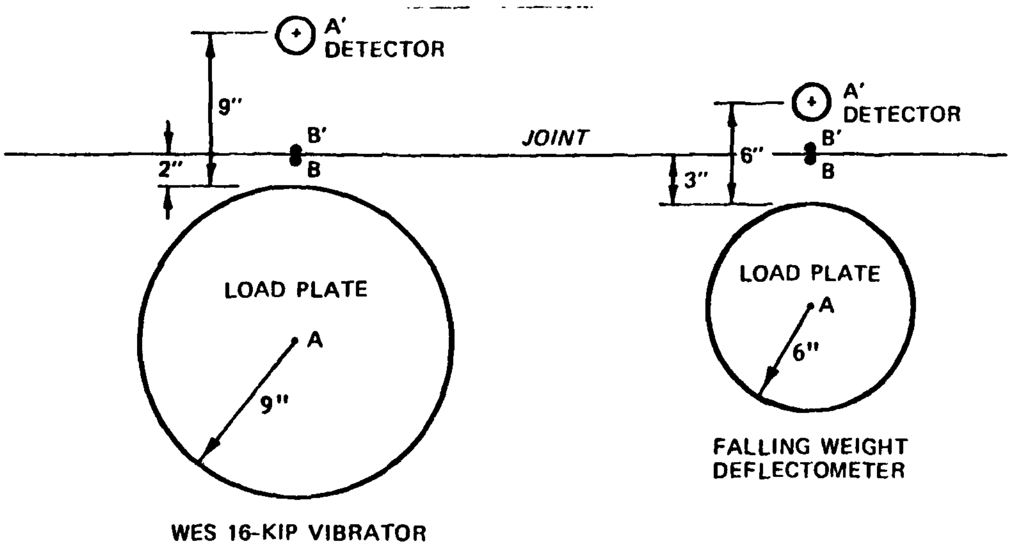


Figure 22. Layouts of NDT tests

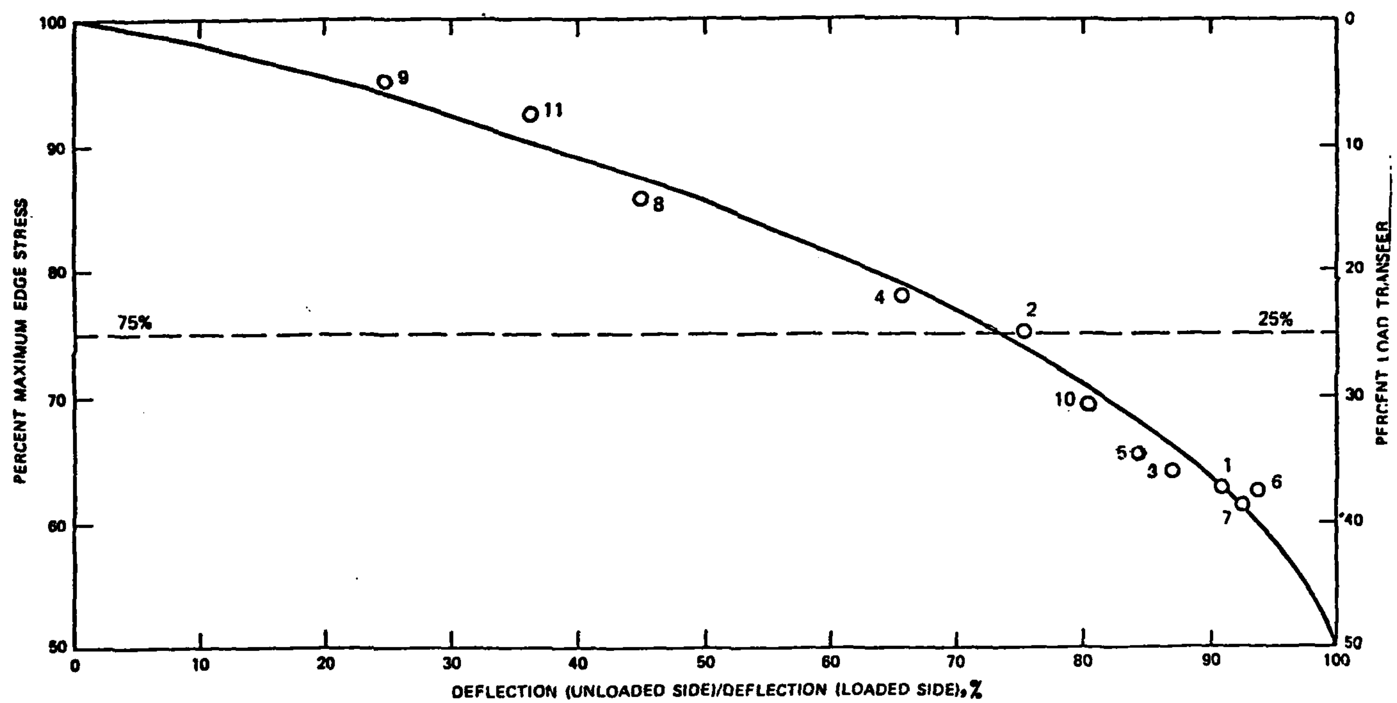


Figure 23. Determination of joint efficiency from measured deflections

Table 13

Pavement Data and Loading Conditions

Number	Loading	Pavement Thickness in.	Subgrade k pci	Concrete Modulus psi	Dowel Bar Diameter in.
1	WES 16-kip	10.5	200	4,000,000	3.0
2	WES 16-kip	10.5	200	4,000,000	1.0
3	WES 16-kip	10.5	400	4,000,000	3.0
4	WES 16-kip	10.5	400	4,000,000	1.0
5	WES 16-kip	10.5	400	3,000,000	3.0
6	WES 16-kip	20.0	200	4,000,000	3.0
7	WES 16-kip	20.0	400	3,000,000	3.0
8	WES 16-kip	20.0	100	4,000,000	1.0
9	WES 16-kip	20.0	100	3,000,000	0.5
10	FWD	10.5	400	4,000,000	3.0
11	FWD	20.0	200	4,000,000	0.5

is only one magnitude of load used in computation, the relationship is independent of the load, as long as the load is small enough that the stresses in the concrete slab are within the elastic range.

It is recommended that additional provisions be added to Chapter 4, paragraph 74, in the existing FAA Advisory Circular as follows.

74. JOINTING OF CONCRETE OVERLAYS.

f. The design procedure assumes a 25 percent edge stress reduction resulting from good load transfer, i.e., 25 percent load transfer. The percent load transfer is defined to be the stress in the unloaded side of the joint divided by the sum of the stresses in the loaded and unloaded sides, multiplied by 100. It is recommended that measurements be made of the actual load transfer capability of the joints of the existing concrete slab to be overlaid if the existing joint condition seems to be deteriorated. If the measured load transfer is well below the specified 25 percent, the overlay can be designed either by using mechanical devices or other means or based on higher flexural stress in the overlay due to poor load transfer in the base concrete slab. The determination should be based on economic considerations. NDT equipment can be used to determine the percent load transfer across the joint based on the measured deflections. The relationship shown in Figure 23 can be used for this purpose. The locations of the measuring plates and the detectors are shown in Figure 22 and should be used in the measurement.

IMPROVED CONDITION SURVEY METHODS

The current FAA Advisory Circular on overlay design provides inadequate guidance for performance of condition surveys of existing pavements. A comprehensive survey and evaluation of distress and other items (such as surface drainage) are essential to the successful design of an overlay (or any other rehabilitation alternative). The PCI is recommended for use in evaluating pavement conditions and establishing the failure criteria for both PCC and AC overlays.[21-28] In so doing, an equivalent performance for flexible and rigid overlays may be obtainable. It is strongly recommended that the PCI method be immediately implemented, but on a trial basis, to begin to accumulate information on both the condition of pavements in service and on the effectiveness and suitability of the method. The procedure outlined in reference 27 can provide the basis for implementation of the system. Since the PCI

method is fairly new to pavement evaluation practices, accumulation of experience with the method is of vital importance in verifying the general validity of the method.

Concerning the implementation of the PCI method, Ahlvin commented that the method has enjoyed ready acceptance because of the concepts involved and the promise it offers for a numerical measure of pavement deterioration. The method has not, however, been exercised sufficiently as a continuing assessment of in-service pavements to verify the general validity of the method or the inherent quantifications of distress elements in relation to one another or between pavement types.

DEFINITION OF FAILURE

A lengthy discussion was given previously in this report on the subject of failure criteria in pavement design. The conclusion was that a different failure criterion was used in concrete and bituminous overlays and bituminous overlays had a PCI approximately 20 points lower than the concrete overlay over a 20-year period even though the stress in the underlying slab is theoretically equal. Since the PCI method is used to evaluate pavement conditions, a more consistent definition of failure for both types of overlays can be expected from it. It should be reiterated, however, that the use of PCI method will change the existing overlay failure criteria; i.e., concrete overlays have been considered failed at the first signs of cracking, while bituminous overlays have been considered failed after considerable cracking and rutting has developed. The proposed failure criteria will either increase the thickness of bituminous overlays or decrease the thickness of concrete overlays.

VERIFICATION AND MODIFICATION OF DESIGN EQUATIONS

The thickness design equations in the current FAA Advisory Circular were developed by the U. S. Army Corps of Engineers primarily through field experiments. In the design methodology, the thicknesses which are determined are based on "deficiency" type analyses. That is, an overlay thickness is determined as the difference between a new pavement thickness required for the new loading and the thickness of the existing pavement adjusted to reflect its current condition. The adjustment factors are termed C_r for concrete overlays and C_b and F for bituminous overlays. The equations for concrete overlays also contain an exponent n, which is dependent on the bond

condition between the new and existing pavement. The design equations for bituminous and concrete overlays are Equations 1 and 3, respectively.

Since the PCI method is proposed for use in evaluating pavement conditions, the significance of C_r and C_b will be modified in the phase II study. The remaining factors to be modified are the F-factor for bituminous pavements and the exponent n for concrete pavements. These are discussed in the following paragraphs. Equation 18 developed by Monismith, Yücë, and Finn[5] has been suggested to replace the bituminous overlay design equation in the existing FAA Advisory Circular. The selection of a value of 0.6 for C_b in Equation 18 removes any consideration of base slab condition for design and would deemphasize any need for better means of selecting the C_b value. It is therefore suggested to use a variant for C_b instead of a constant value of 0.6. Also for convenience of design practice, the coefficient 0.3 in Equation 18 is changed to 0.33. Since Equation 18 yields a thicker overlay than the existing equation (Equation 1) at a higher coverage level and since the field experience of the FAA indicates that concrete overlay performs better than bituminous overlay, the use of Equation 17 to replace the existing equation should tend to make the two overlay designs more nearly equivalent in terms of long-term performance.

Since a granular base course is not allowed in nonrigid overlay in the FAA overlay design procedure, the term $0.34t_2$ in Equation 17 and the term $0.255t_2$ in Equation 18 can be omitted. To adopt Equation 17 for bituminous overlay design, Chapter 4, paragraphs 67b, 67c, and 67d of the existing FAA Advisory Circular should be revised and combined into new paragraphs 67b and c as follows.

67. <u>BITUMINOUS OVERLAY ON EXISTING RIGID PAVEMENT.</u>

*b. The thickness of the bituminous overlay is computed from the following formula:

$$h = \frac{1}{F} (0.33t + C_b\, h_e)$$

or

$$t = 3.0 (Fh - C_b\, h_e)$$

where

h = single thickness of rigid pavement required for design conditions, inches

F = factor which controls the degree of cracking in the base pavement

t = thickness of overlay bituminous layer, inches

h_e = thickness of existing rigid pavement, inches

C_b = condition factor for the existing concrete slab. It was 0.6 in the development of the equation. Values higher than 0.6 may be considered if deemed appropriate.

c. The design of a bituminous overlay for a rigid pavement which has an existing bituminous overlay is slightly different. The designer should treat the problem as if the existing bituminous overlay were not present, calculate the overlay thickness required, and then adjust the calculated thickness to compensate for the existing overlay. If this procedure is not used, inconsistent results will often occur.

(1) An example of the procedure follows. Assume an existing pavement consists of a 10-inch (25 cm) rigid pavement with a 3-inch (7.5 cm) bituminous overlay. The existing pavement is to be strengthened to be equivalent to a single rigid pavement thickness of 14 inches (36 cm). Assume an F factor of 0.9 and C_b of 0.6 are appropriate for the existing conditions.

(2) Calculate the required thickness of bituminous overlay as if the existing 3-inch (7.5 cm) overlay were not present.

$$14 = \frac{1}{0.9}\,(0.33t + 0.6 \times 10)$$

$$t = 20 \text{ inches } (50.8 \text{ cm})$$

(3) An allowance is then made for the existing bituminous overlay. In this example assume the existing overlay is in such a condition that its effective thickness is only 2.5 inches (6 cm). The required overlay thickness would then be 20 - 2.5 = 17.5 inches (44.6 cm). The determination of the effective thickness of the existing overlay is a matter of engineering judgement.*

The only difference in the design overlay thickness between the existing and the proposed equations is the difference in the coefficient 2.5 (for the existing equation) and 3.0 (for the proposed equation). In other words, the overlay thickness designed by the proposed equation is 20 percent more than that designed by the existing equation. It is worth pointing out that in the current FAA Advisory Circular, paragraph 67e states:

> The formula for calculating the thickness of bituminous overlays on rigid pavements is limited in application to overlay thicknesses which are equal to or less than the

> thickness of the base rigid pavement. If the overlay thickness exceeds the thickness of the base pavement, the designer should consider designing the overlay as a flexible pavement and treating the existing rigid pavement as a high quality base material. This limitation is based on the fact that the formula assumes the existing rigid pavement will support considerable load by flexural action. However, the flexural contribution becomes negligible for thick bituminous overlays.

Since the proposed equation will increase the overlay thickness 20 percent, the possibility that the overlay thickness will exceed the thickness of the base concrete slab will be higher, and the designers should be aware of this situation.

In analyzing Corps of Engineers' traffic test data, Monismith, Yücë, and Finn[5] found that there was insufficient information to derive any meaningful results for cases of bonded and partially bonded concrete overlays. The revised equation (Equation 20) for unbonded overlays was developed based only on nine test items. Table 11 shows that the overlay thicknesses determined from Equation 20 are much greater than those determined from equations developed by Chou[9] and Lytton,[8] respectively. Since Equation 20 was modified based on a very limited amount of data, it is felt that Equation 20 developed by Monismith should not be adopted to replace the existing equation at the present time, and further research work should be done in phase II study in this area.

In the Corps of Engineers' overlay design procedure for military airfields, modified equations for concrete overlays are used when the difference in the flexural strengths of the overlay concrete and the existing base slab is greater than 100 psi. Because this principle is analytically sound, it is recommended that the FAA adopt the same modification. The following change should be made in Chapter 4, paragraphs 72a and b in the FAA Advisory Circular.

72. a. Concrete Overlay Without Leveling Course. The thickness of the concrete overlay slab applied directly over the existing rigid pavement is computed by the following formula:

$$h_c = \sqrt[1.4]{h^{1.4} - C_r h_e^{1.4}}$$

h_c = required thickness of concrete overlay

h = required single slab thickness determined from design curves

AD-A135 317 INVESTIGATION OF THE FAA OVERLAY DESIGN PROCEDURES FOR RIGID PAVEMENTS(U) ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG MS GEOTE.. Y T CHOU AUG 83 2/2

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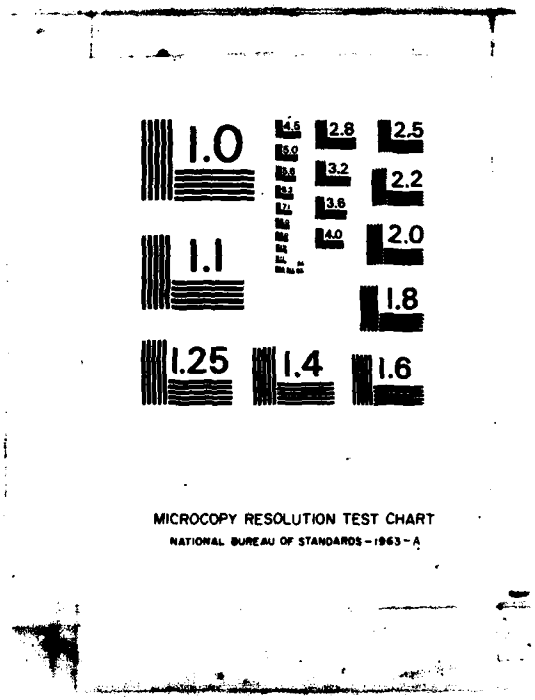
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3.2
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4.0
MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

h_e = thickness of existing rigid pavement

C_r = condition factor

Due to the inconvenient exponents in the above formula, graphic displays of the solution of the formula are given in Figures 4-9 and 4-10. These graphs were prepared for only two different condition factors, C_r = 1.0 and 0.75. The use of a concrete overlay pavement directly on an existing rigid pavement with a condition factor of less than 0.75 is not recommended because of the likelihood of reflection cracking. *The above equation assumes the flexural strength of the concrete used for the overlay will be approximately equal to that of the base pavement. When such is not the case and the flexural strengths differ by more than 100 psi, the following modified equation will be used to determine the required thickness of the overlay

$$h_c = \sqrt[1.4]{h^{1.4} - C_r \left(\frac{h}{h_b} h_e\right)^{1.4}}$$

h_b = required single slab thickness determined from design curves based on the flexural strength of the base pavement.*

b. Concrete Overlay with Leveling Course. In some instances it may be necessary to apply a leveling course of bituminous concrete to an existing rigid pavement prior to the application of the concrete overlay. Under these conditions a different formula for the computation of the overlay thickness is required. When the existing pavement and overlay pavement are separated, the slabs act more independently than when the slabs are in contact with each other. The formula for the thickness of an overlay slab when a leveling course is used is as follows:

$$h_c = \sqrt{h^2 - C_r h_e^2}$$

h_c = required thickness of concrete overlay

h = required single slab thickness determined from design curves

C_r = condition factor

h_e = thickness of existing rigid pavement

*When the flexural strength of the overlay and of the existing pavements differ by more than 100 psi, the equation is modified as follows:

$$h_c = \sqrt{h^2 - C_r \left(\frac{h}{h_b} h_e\right)^2}$$

h_b = required single slab thickness determined from design curves based on the flexural strength of the base pavement.*

If these two modified equations are used, Chapter 4, paragraph 64 of the existing FAA Advisory Circular should be modified accordingly:

64. MATERIAL SELECTION CONDITIONS. Criteria are presented in this chapter for both bituminous and concrete overlay pavements. The selection of the overlay type should be made after careful consideration of many factors. The designer should consider the total life-cycle cost of the overlay pavement. Life-cycle costs should include initial construction and maintenance costs over the design life of the pavement. Other considerations such as allowable down time of the pavement and availability of alternate pavements to use during construction will have a significant impact on the overlay type selected. *For overlay design, the concrete flexural strength should be properly selected based on the following criteria:

Type of Overlay	Design Flexural Strength
Bituminous concrete	Existing concrete slab
Fully bonded concrete	Existing concrete slab
Partially bonded concrete	Existing and overlay concrete slabs
Unbonded concrete	Existing and overlay concrete slabs

The actual value of flexural strength of the existing slab should be determined from either (1) coring the slabs at several locations (20 cores minimum) and then testing for indirect tensile strength (and then transposing this strength into flexural strength), or (2) cutting standard sized beams from a few slabs and directly measuring their flexural strength. Once the mean and standard deviation of the flexural strengths are computed, the design flexural strength can be determined from the equation shown in paragraph 44a.*

ANOMALIES OF THE DESIGN PROCEDURES

The possible causes of anomalies in overlay designs were discussed earlier. It is suggested that a new paragraph be added to Chapter 4 of the existing FAA Advisory Circular as follows.

*78. <u>POSSIBLE ANOMALIES</u>

Because of the difference in the failure mechanism in flexible and rigid pavements and also because of the difference in design concept between the flexible and rigid overlays, conflicting results may be obtained in some design conditions using the suggested overlay design procedures. These cases sometimes occur with strong subgrade soil or with existing composite pavements, i.e., flexible over rigid pavement. Engineering judgment should be exercised to determine the best type of overlay for the particular pavement. The more economical design can be selected for use.*

The condition factor C_b in the bituminous overlay ranges from 1.0 to 0.75. A lower value for C_b represents a severely cracked base slab, which is not recommended because of the likelihood of severe reflection cracking. However, severely cracked base slabs are often used as a base course when the conventional flexible pavement procedure is used for the overlay design but the practice is not recognized in the FAA Advisory Circular. It is suggested that a new paragraph be added to paragraph 67 of the existing FAA Advisory Circular as follows.

67. <u>BITUMINOUS OVERLAY ON EXISTING RIGID PAVEMENT.</u>

f. The condition factor C_b in the bituminous overlay ranges from 1.0 to 0.75. A lower value for C_b represents a severely cracked base slab, which is not recommended because of the likelihood of severe reflection cracking. However, severely cracked base slabs can be used as base course when conventional flexible pavement procedure is used for the overlay design.

IMPACTS OF PROPOSED IMPROVEMENTS

The impacts on the design thickness of overlays due to the various proposed improvements are summarized in the following table.

Table 14

Impact of Proposed Improvements on Design Thickness of Overlays

Proposed Improvements	Impact on Design Thickness of Overlays	
	Asphaltic Concrete	Portland Cement Concrete
Uniform procedures to determine the design flexural strength of the PCC and k-values	None	None
Use of nondestructive testing (NDT) to determine foundation k-values	None	None
Use of NDT to detect voids under the slab	None	None
Measurement of load transfer	None	None
Improved condition survey method	None or Increase	None or Decrease
Definition of failure	None or Increase	None or Decrease
Verification and modification of design equations	Increase	None
Anomalies of the design procedures	None	None

PROPOSED PHASE II STUDY

A number of programs were outlined in the Board of Investigators' report for the long-range improvement of the FAA overlay design method; details can be found in Appendix B. The recommended programs are combined in the following three work units to be investigated in Phase II of this study. The results will provide an overlay design package suitable for entry into the FAA AC.

IMPROVED CONDITION SURVEY METHODS

The current FAA Advisory Circular on overlay design provides inadequate guidance for performance of condition surveys of existing pavements. A comprehensive survey and evaluation of distress is essential to the successful design of an overlay. The PCI[20-27] is recommended for use in evaluating pavement conditions and establishing the failure criteria for both concrete and bituminous overlays. The values of C_b and C_r in Equations 1 and 3 will be evaluated based on the rated PCI values. In so doing, an equivalent performance for flexible and rigid overlays becomes more likely.

OVERLAY DESIGN USING LAYERED ELASTIC THEORY

Design procedures based on layered elastic theory for both flexible and rigid airport pavements have been developed at WES.[18,37] Efforts are being made to implement these procedures into Corps of Engineers design manuals. In this earlier work with elastic layered design methods, it was noted that an elastic layered method of overlay design could be developed.[18] The major problems that have to be solved include:

a. The selection of analytical interface conditions to represent the degree of field bonding between the overlay and base pavement.

b. The techniques to quantitatively characterize the load deformation characteristics of cracked base pavements.

The development of the layered elastic overlay design procedure would take the same approach as was taken in development of the design procedure for new rigid airport pavements; i.e., test pavement data would be used to develop relationships between computed parameters and performance. The parameters to be considered in the analysis of the data would be stresses at the bottom of the overlay and of the base slab. The variables that could be considered in such a design procedure are material properties, applied load, layer thickness,

bonding between layers, past traffic, and base slab condition. Since the layered elastic computer program has the capability of considering different degree of bonding conditions between pavement layers, the effect of different bonding conditions on overlay thickness can be incorporated into the design procedures.

CONSIDERATION OF MATERIAL VARIABILITIES IN THE OVERLAY DESIGN

A critical factor in the design of overlays is to determine the level of reliability required of the overlay. In technical terms, reliability is the probability that a pavement will not fail. The factors that cause reliability to drop are large variabilities in material properties, layer thicknesses, and projected air traffic volumes. Research projects to consider the material variabilities in both the flexible and rigid pavement design procedures are currently going on at WES. The degree of material variability can be input and pavement thicknesses can be selected based on desired reliabilities of the design. Similar procedures can also be used to consider the material variabilities in the new overlay design procedure.

VALIDATION OF THE NEW OVERLAY DESIGN METHODOLOGY

Performance data for overlays in five airports pavements throughout the United States will be collected and studied to validate and improve the new overlay design procedure.

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APPENDIX A: CORPS OF ENGINEERS OVERLAY DATA

By Ray Rollings

1. Overlay Apron Pavement, Alexandria, Louisiana

1.1 Objective: Evaluate performance of rigid overlays under traffic of B-17.

1.2 Overlay Test Items: 1 rigid overlay.

1.3 References:

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2. Pavement Overlays, MacDill Field, FL

2.1 Objective: Determine thickness of flexible and rigid overlay to support 3000 coverages of 60,000 lb dual-wheel loading.

2.2 Overlay Test Items: 5 flexible and 2 rigid overlays.

2.3 References:

2.3.1 "Report on Accelerated Traffic Tests on Pavement Overlays at MacDill Field, Florida," U. S. Eng Office, Savannah, GA.

3. Hamilton Field Concrete Pavement Investigation

3.1 Objective: Aid in design of concrete overlay for heavy plane loads.

3.2 Overlay Test Items: 11 rigid overlays.

3.3 References:

3.3.1 "Final Report on Concrete Pavement Investigations at Hamilton Field," Corps of Engineers, South Pacific Division, Dec 1946.

3.3.2 "Report on Instruments Used on Test Track at Hamilton Field," Corps of Engineers, South Pacific Division, Dec 1946.

4. Lockbourne No. 1

4.1 Objective: Study effects of types of subgrade, type and thickness of granular base, type and spacing joints, steel reinforcement for rigid pavements and to study behavior of concrete overlays.

4.2 Overlay Test Items: 10 rigid overlays.

4.3 References:

4.3.1 "Design and Construction Report Lockbourne Test Track," Ohio River Division Laboratory, June 1944.

4.3.2 "Lockbourne No. 1 Test Track Final Report," Ohio River Division Laboratory, March 1946.

4.3.3 "Lockbourne No. 1, Test Track Lockbourne Army Air Base - Photographs."

4.3.4 "Report of Reconstruction Lockbourne Test Track," Ohio River Division Laboratory, Jan 1945.

5. Lockbourne No. 2 Experimental Mat

5.1 Objective: Investigate effect of slab thickness, subgrade type steel reinforcement, bases, joints, and overlay construction under traffic of 150-kip wheel load.

5.2 Overlay Test Items: 6 rigid overlays.

5.3 References:

5.3.1 "Lockbourne No. 2,300,000 Pound Experimental Mat, Report of Construction," Ohio River Division Laboratory, June 1945.

5.3.2 "Final Report Lockbourne No. 2 Experimental Mat," Ohio River Division Laboratory, May 1950.

6. Lockbourne No. 3

6.1 Objectives: Determine design criteria for flexible overlays for B-29 (120 kip gross load).

6.2 Overlay Test Items: 8 flexible overlays.

6.3 References:

6.3.1 "Lockbourne No. 3 - Overlay Mat Pavement Overlay Investigation, Report of Construction," Ohio River Division Laboratory, June 1948.

6.3.2 "Lockbourne No. 3 - Pavement Overlay Investigation, Final Report," Ohio River Division Laboratory, May 1951.

7. Sharonville Overlay Studies

7.1 Objective: Develop flexible and rigid overlay criteria.

7.2 Overlay Test Items: 8 rigid and 38 flexible overlays.

7.3 References:

7.3.1 "Overlay Test Track, Sharonville, Ohio, Report of Construction," Ohio River Division Laboratory, September 1954.

7.3.2 "Specifications for Construction of Overlay Test Track, Sharonville Engineer Depot, Sharonville, Ohio," Huntington District, Corps of Engineers, May 1951.

7.3.3 "Subgrade Preparation for Overlay Test Track No. 2, Sharonville, Ohio," U. S. Army Engineer Waterways Experiment Station, Miscellaneous Paper No. 4-47, August 1953.

7.3.4 "Specifications for Construction of Overlay Test Track No. 2," Huntington District, Corps of Engineers, June 1953.

7.3.5 "Photographs of Sharonville Nos. 1 and 2" Ohio River Division Laboratory, n.d.

7.3.6 "Overlay Test Track, Sharonville, Representative Photographs," Ohio River Division Laboratory, n.d.

7.3.7 "Report of Special Overlay Study Test Track No. 3, Sharonville, Ohio," Ohio River Division Laboratory, September 1954.

7.3.8 "Minutes of Board of Consultants Meeting Sharonville Overlay Pavement Investigation, 29 November 1954," Ohio River Division Laboratory, 20 Dec 1954.

8. <u>Heavy-Load Test Tracks, Sharonville, Ohio</u>

8.1 Objective: Verify design criteria for 325 kip twin-tandem gears and obtain experimental data on rigid overlays.

8.2 Overlay Test Items: 5 rigid overlays.

8.3 References:

8.3.1 "Heavy-Load Test Tracks, Report of Construction," Ohio River Division Laboratory, Tech Report 4-17, Feb 1961.

8.3.2 Weekly Progress Reports 20 Feb 1957 - Sep 1959, Ohio River Division Laboratory.

8.3.3 Heavy Load Investigation Consultants Briefing by T. R. Walthen, Ohio River Division Laboratory, Feb 1959.

8.3.4 "Specifications for Construction of Heavy Load Test Tracks at Sharonville, Ohio," Huntington District, Corps of Engineers, May 1957.

8.3.5 Minutes, Board of Consultants Meeting, 12 Feb 1959.

8.3.6 Ohio River Division Laboratory Participation in Joint Conference on Military Investigational Programs, 1958.

9. <u>Multiple-Wheel Heavy Gear Load Tests</u>

9.1 Objective: Develop design criteria on aircraft with gross load above 600 kips.

9.2 Overlay Test Items: 2 flexible overlays.

9.3 Reference:

9.3.1 "Multiple-Wheel Heavy Gear Load Pavement Tests," U. S. Army Engineer Waterways Experiment Station, Technical Report S-71-17, 4 Volumes, November 1971.

10. <u>Model Tests</u>

10.1 Objective: Study overlay behavior.

10.2 Original data available - not published.

11. <u>Other Publications</u>

11.1 Mellinger, F. M., and Sale, J. P., "The Design of Non-Rigid Overlays for Concrete Airfield Pavements," <u>Journal Air Transport</u> Division, V82, AT2, American Society of Civil Engineers, May 1956.

11.2 Sale, J. P., and Hutchinson, R. L., "Development of Rigid Pavement Design Criteria in Military Airfields," <u>Journal Aerospace Transport Division,</u> V88, No. AT1, American Society of Civil Engineers, August 1962.

11.3 Mellinger, F. M., "Structural Design of Concrete Overlays," Presented at Committee 325 Symposium on Highway Problems at the 58th Annual Convention, American Concrete Institute, Denver, CO, March 1962.

APPENDIX B: INVESTIGATION OF UPDATE OVERLAY THICKNESS CRITERIA

INTRODUCTION

The Board of Investigators convened at the U. S. Army Engineer Waterways Experiment Station (WES), Vicksburg, Mississippi, on 9-10 February 1982 for the purpose of discussing overlays for rigid pavements and making recommendations to the Federal Aviation Administration (FAA). The objective of the investigation was to identify shortcomings in the present FAA overlay design procedure, determine methods of improving overlay design, and recommend research required to develop and validate an airport overlay design procedure that would yield equivalent performance for flexible and rigid overlays on a rigid base pavement. The FAA Technical Representative for this investigation was Dr. Aston McLaughlin.

Besides the engineers from WES, the Board is comprised of the following members:

a. Professor Carl Monismith, The University of California at Berkeley.

b. Professor Michael Darter, The University of Illinois.

c. Professor Robert Lytton, Texas A&M University.

d. Professor Walter Kilareski, Pennsylvania State University.

Prior to the conference, each Board member studied the Corps of Engineers' overlay test data provided by WES and prepared a report. The reports were submitted to WES and duplicate copies of each report were made and sent to other members for review.

During the Conference, each Board member presented a summary of the highlights of his investigation and recommendations, followed by answering questions and discussions. At the completion of the Conference, the Board of Investigators prepared a combined report to the FAA based on the recommendations and the concensus of the discussions.

The report is as follows and is made up of two parts:

a. Short-term improvements.

b. Long-term programs.

SHORT-TERM IMPROVEMENTS

Several serious deficiencies have been identifie in the FAA overlay design procedure.[1] There are existing procedures and technology that can be added relatively quickly and easily that will greatly improve the procedures. These recommended improvements are called "short-term" in that they do not require a major research and development effort and could be implemented within the next two years. Other improvements that will require a major research and development effort are called "long-term" and may require several years.

The following "short-term" improvements are considered to be very important and should be undertaken immediately to upgrade several deficiencies in the FAA Advisory Circular.

a. Selection of appropriate "failure" level (or terminal condition) for flexible overlays of rigid pavements.

b. Improved condition survey methods.

c. Verification and modification of the n, F, C_b, and C_r coefficients in the existing equations to reflect current traffic and conditions.

d. Incorporation of deflection measurements for load transfer, void detection, and k-value estimation.

e. Improvements for load transfer where it is determined to be deficient.

f. Uniform procedures to determine the "design" flexural strength of the PCC.

g. Identification of some apparent limitations and anomalies in the current design procedure.

h. Use of life-cycle cost methods in consideration of alternate overlay designs.

i. Possible inclusion of recent Corps of Engineers structural design procedures using elastic layer theory.

j. Increased emphasis on evaluating and correcting subsurface drainage problems before the overlay is placed.

DEFINITION OF FAILURE

The current FAA Advisory Circular[1] is a compilation of over 30 years of research, development, and field verification. Over these years there has not been a consistent definition of overlay pavement failure. Rigid pavement overlays have been considered failed at the first signs of cracking while asphaltic-concrete pavement overlays have been considered failed after

extensive cracking and rutting developed. It is improper to assume that the performance of the two pavement types at the "failed" condition is the same. The designer, however, is led to believe that the two overlay designs, suggested in the Circular, will perform equally (i.e., have the same functional condition at failure).

The inconsistent definition of failure has been incorporated into the overlay design procedure. In other words, the development of the procedure worked towards two different "failure" conditions; but, the Circular does not provide any guidance or background in this area. Therefore, the designer will often calculate two designs which do not represent equal or reasonable sections when analyzed by engineering experience.

The Board is of the opinion that the design procedure can be improved if the FAA would define the "failed" condition of both a portland cement overlay and an asphaltic-concrete overlay to minimize the possibility of inconsistencies in the designed results. If the failure criteria are not redefined, i.e., the coefficients n, F, C_b, and C_r are not properly adjusted, the Board recommends that areas in which inconsistencies can result be identified and pointed out in the AC. Guidance in what procedure to follow when inconsistencies occur should be delineated in the AC.

CONDITION SURVEY

The current FAA Advisory Circular on overlay design provides inadequate guidance on performing a condition survey of the existing pavement. A comprehensive survey and evaluation of distress and other items (such as surface drainage) is essential to the successful design of an overlay (or any other alternative).

Distress data can be used to determine the major causes and mechanisms causing pavement deterioration. Distress data are also used to select the condition factors (C_b , C_r), joint sealant condition, repair work required before overlay, and to assess the need for other pavement evaluation tests (e.g., subdrainage, joint load transfer, void detection).

The condition survey could also include other items that have impact on the design of the overlay that can be accomplished at the same time such as surface drainage problems.

WES made a review for the FAA of available condition survey procedures (FAA-RD-80-55, "Procedure for Condition Survey of Civil Airports," May 1980)

and recommended the Pavement Condition Index (PCI)[2-8] for use on civil airports. The PCI method was developed for the Air Force by the Construction Engineering Research Laboratory (CERL) and has been implemented by the Air Force (AFR 93-5). The PCI was introduced to civil airport engineers through a series of seminars and met with enthusiastic acceptance.

VERIFICATION AND MODIFICATION OF EQUATION FACTORS

The current FAA Advisory Circular paragraphs which pertain to overlay design procedures were developed by the Army Corps of Engineers through field experiments (test tracks). The thickness cross-section of an overlay is based upon a "deficiency" type design. This involves the selection of a new pavement structure thickness and evaluation of the old pavement thickness. A difference in the two thicknesses represents the required overlay thickness. Since the condition of the existing pavement represents its structural capabilities, the thickness is modified to provide an "equivalent" thickness of new pavement. The C_r factor in rigid overlay design and the C_b factor in asphalt overlay design as well as the F factor are used to adjust the pavement section requirements.

The form of the equation for rigid overlays, which is:

$$h_c = (h^n - C_r h_e^n)^{1/n}$$

also has the exponent n which is used to determine whether there is full bond, partial bond, or no bond between the slabs.

The factors in these equations have been empirically derived from the test track data. The Board has reservations whether the factors are both accurate and realistic, for example, the C_r factor can range from 0.35 to 1.0 while the C_b can vary from only 0.75 to 1.0. It appears the value for the F factor must always be 0.90 to 1.0 in the range of k values and number of departures. The derivation of these numbers and why the range of numbers was selected appear to have been lost over the years.

Another problem is that the designer must "subjectively" select the value for the factor. The Advisory Circular does provide some guidance in the selection of the factor; however, the guidance provided is only limited. Consequently, the designer can easily select the wrong factor. The Board recommends that the FAA reevaluate the derivation of the C_b, C_r, F, and n factors in the design equation. This would involve a careful examination of

the test track data, especially since some of the data have been lost and existing data are questionable. Test track loadings, material properties, coverages to failure, and environment must be heavily scrutinized to determine their usefulness. It is suggested that the test track data be supplemented with in-service pavement data if possible. This could provide realistic loadings as well as environmental effects that are not incorporated in test track-type data.

The Board suggests also that the FAA seriously consider the design approach suggested by Dr. Lytton. His derivation of the overlay equation has been based upon a fracture mechanics approach, and as such, provides a mechanistic derivation of the equation form. This type of approach can be supported with sound engineering principles; and, after the form of the equation has been established, the validation of the coefficients and exponents can be accomplished through the use of existing data in conjunction with limited new testing.

DEFLECTION MEASUREMENTS (NDT)

The FAA Advisory Circular on overlay design provides inadequate guidance on the use of NDT and its valuable role in overlay design. The advent of the heavy wide-bodied aircraft has increased the problem occurring at joints. Some of these problems include poor load transfer across the joint and the permanent deformations beneath the slab corners from large deflections.

Many airport pavements do not have adequate load transfer across the joints as is assumed in the FAA design procedure. This results in very high stresses and deflections which lead to cracked slabs. This has occurred, for example, at O'Hare in asphalt overlays of concrete pavements. The stress on the subgrade is very high at corners having poor load transfer. Heavy load NDT can be used to measure joint load transfer. The measured deflection transfer must be converted to stress transfer to see if the FAA 25 percent-assumed reduction is met. WES has demonstrated the capability to perform joint-efficiency evaluations with the WES 16-kip vibrator.

Voids have developed under joints and corners of concrete slabs. These voids are caused either from pumping of the foundation or permanent deformation of the foundation. Heavy load NDT can be used to detect voids beneath corners and edges of concrete slabs. WES has experience in this area also.

The k-value test is difficult, expensive, and time consuming. It also is only performed at one or two locations on a pavement facility (e.g., RW). NDT can be easily used to determine an elastic repeated load k-value at the interior of a slab by back calculation through standard Westergaard-type equations or influence charts. The repeated load k-value can then be reduced to the standard gross k-value through an empirical factor. Some work is required to determine the difference in k-value obtained between different NDT devices.

The FAA new design charts assume 25 percent edge stress reduction from good load transfer (this is equivalent to about 75 percent deflection transfer). If an existing pavement does not have at least 75 percent deflection transfer, the overlay will be underdesigned.

It is recommended that the AC be modified to include the following:

a. The measurement of deflection transfer across selected joints be required using heavy load devices. The new design single-slab thickness should be modified to include lower levels of load transfer across a joint.

b. A statement requiring the assessment of whether or not voids exist beneath the joints and corners and that corrective measures be taken if voids exist (e.g., subsealing, subdraining). Heavy-load NDT should be allowed as one means of determining if voids exist.

c. The use of heavy-load NDT to back calculate full support k-values at the slab center be allowed in lieu of the standard plate bearing test.

LOAD TRANSFER IMPROVEMENTS

The FAA Advisory Circular on overlay design is deficient in addressing the problem of load transfer at joints. Poor load transfer is a very serious problem and can lead to pumping, joint spalling, and slab breakup. The FAA overlay design assumes good load transfer in the existing pavement. If there is poor load transfer, the overlay may be underdesigned.

Heavy-load NDT equipment is available (WES 16-kip, FWD, heavy road-rater, actual aircraft, or other load vehicles with special measurement devices to measure joint and crack load transfer). Several such studies have been conducted on existing airport pavements where joint deficiencies have been identified.

The problem of restoring load transfer at joints was studied at WES in a project jointly funded by FAA, OCE, and USAF. The results of the study are reported in FAA-RD-72-106 (WES MP-S-72-43). Recently mechanical devices have

become available for improving load transfer at joints and have been installed in airport pavement with success.

It is recommended that the AC[1] include a section that explains the importance of good load transfer, how to measure it, and allows the consideration of the installation of mechanical devices where needed.

SELECTION OF FLEXURAL STRENGTH

An important input required for the overlay design procedure is the selection of the flexural strength for the existing portland cement concrete. The FAA Circular suggests that beams be taken from the existing pavement and tested. The Circular does not provide any guidance for the selection of the modulus of rupture value based upon the variability of the material. Since the flexural strength of concrete varies a great deal, the value selected will have a direct effect on the calculated stresses in the pavement resulting in a wide range of overlay thicknesses.

The Board recommends that the FAA develop a statistically sound procedure for the selection of the modulus of rupture which is used in the overlay design. It is also recommended that guidance in what procedure to follow when the flexural strength of the existing concrete slab is different from that of the overlay be provided in the AC.

LIFE-CYCLE ANALYSIS

The current FAA overlay design assumes that both asphalt concrete and portland cement concrete have equivalent performance. As such, an initial capital cost analysis can be done for the economic selection of the pavement system. As was pointed out earlier, the Board is of the opinion that a redefinition of failure is needed. This would invalidate the current economic analysis scheme. With a redefinition of failure, it must be recognized that there will be a difference in the design periods for the portland cement concrete and the asphalt concrete overlays. There will also be a difference in the type and level of maintenance required for each pavement system. Therefore, it appears that an economic analysis based upon initial costs would be misleading.

Also of concern are the user costs associated with runway closing at major airports for either routine maintenance or extreme rehabilitation. The current approach does not take this into account. Other rehabilitation

schemes such as recycling should also be considered in the economic analysis since they do represent a "salvage" value of the pavement.

The Board, therefore, recommends that the FAA adopt an economic analysis which would incorporate life-cycle costs rather than initial capital costs.

OVERLAY DESIGN BASED ON LAYERED ELASTIC THEORY

Under sponsorship of FAA and OCE, design procedures based on layered elastic theory for both flexible and rigid airport pavements were developed. The design procedure for flexible pavements, reported in FAA-RD-74-199 (WES TR S-75-17) has been implemented as an optional procedure in the Tri-Service Manual for design of military pavements. The design procedure for rigid pavements, reported in FAA-RD-77-81 (WES TR GL-79-4), is in the process of being implemented for design of military airfields. Also under sponsorship of FAA, WES developed a methodology for evaluation of light aircraft pavements. The procedure reported in FAA-RD-80-9-II uses data from NDT and layered elastic theory to evaluate light aircraft pavements.

The development of the layered elastic overlay design procedure would take the same approach as was taken in development of the design procedure for new rigid airport pavements, i.e., test pavement data would be used to develop relationships between computed parameters and performance. The parameters to be considered in the analysis of the data would be stress at the bottom of the overlay and stress at the bottom of the base slab. The variables that could be considered in such a design procedure are material properties, applied load, layer thickness, bonding between layers, past traffic, and base slab condition.

At the present, the advantage of developing such an overlay design procedure would be that it would provide a step toward a more rational procedure, could be quickly developed, would be consistent with new developments in pavement design, and would provide an improvement of the design procedure currently in use.

SUBDRAINAGE

The FAA AC on overlay design provides inadequate guidance on the evaluation of subsurface drainage problems. If excess free water exists beneath a slab and cannot drain out within a short time period, a serious potential exists for pulping and loss of support beneath the slab. This will result in

very high stresses and deflections in the slabs and rapid failure, even under an overlay.

It is recommended that a section be added to the AC that requires the designer to evaluate the adequacy of subsurface drainage and assess the extent of existing deterioration due to moisture problems. If there is a serious problem, the designer should also be required to address this in the design of the overlay by providing subsealing, subdrainage, and/or surface sealing.

LONG-TERM PROGRAM

To insure that existing airfield pavements continue to effectively serve their users we urge that the following program be initiated immediately. In the development of this program we recognize that valuable data and methodology already exist as evidenced by the many successful pavements and overlays in service. This experience should serve as important background for the developments which we recommend. Moreover, the short-term program described in the previous section also will provide valuable information. Thus, this program has a strong basis to insure effective development.

Successful completion of this program will, we believe, insure more effective use of existing materials, respond to changed loading conditions, permit the incorporation of new materials in pavements in a much shorter time frame, readily permit the assessment of construction influences and assist in the development of improved construction specification, and improve the reliability of the design assessment.

Incorporation of the maintenance and rehabilitation techniques which have been recommended for development will serve to insure at specific sites the most effective use of available (and usually limited) funds.

USE OF MECHANISTIC APPROACH

To permit more effective use of existing materials and to permit incorporation of new materials in overlays, we urge that a mechanistic approach be developed making use of finite element methodology. The methodology should be directed to the examination of specific distress modes which occur in overlays and should include both traffic-load associated and non-traffic associated effects (e.g., thermal effects). Further, the model should make use of a representation of the underlying materials as a solid (e.g., the use of elastic moduli) rather than as a dense liquid (i.e., the Westergaard type of foundation) in the mechanistic sense.

Provision should be included in the model for consideration of discontinuities (joints and/or cracks) with or without load transfer capabilities. Load transfer capabilities should permit consideration of special devices (including but not limited to dowels).

The model should permit the prediction of:

a. Fatigue cracking in both asphalt and PCC overlays.

b. Rutting in asphalt-type overlays.

c. Corner settlements from repetitive loading in PCC materials.

d. Reflective cracking in both asphalt and PCC overlays.

The report by Dr. Lytton prepared for the meetings provides an indication of some of the parameters which must be considered in the development of items c. and d. above.

Provision should be included in the model for the use of additional materials to those currently incorporated in the FAA Advisory Circular. For example, the methodology should permit consideration of fabrics (geotextiles) and rubber-asphalts as interlayers; open-graded asphalt concrete, fibrous concrete, continuously-reinforced concrete, prestressed concrete, recycled existing pavement materials, and new materials currently under development.

LONG-TERM PAVEMENT MONITORING

The improvement of overlay design and of the ability to make projections to determine future funding levels for maintenance and rehabilitation activities requires that the equations that are used are realistic and based upon actual field data. Although there have been several excellent research projects that have been conducted in the past to determine pavement response to load, there has not been a corresponding effort at determining the long-term effects of the local climate, aging, and of a less intense loading frequency than is normally used in accelerated load tests on pavements. Also, these research projects have not usually produced equations that can be used in planning for future expenditures of maintenance and rehabilitation funds. The benefit of having long-term performance data is that it can give a basis for making accurate planning estimates and for providing realistic modifications to existing overlay design procedures.

In planning a long-term pavement monitoring effort, it is essential to collect enough data for the purposes intended but also to avoid collecting too much data. A carefully planned experimental design is essential to choose the right size of sample airports on which data are collected. It may be that 30 to 40 airports will be sufficient to provide detailed data for modifying overlay design procedures and for developing planning models which are capable of accurately predicting the remaining life of a pavement given its current condition. The experimental design should include all of the important variables such as traffic level, environment, pavement type, and range of design

thicknesses. This variation can be achieved by carefully selecting airport pavements that are representative of different climatic zones in the United States.

One of the major reasons for collecting such data and developing the equations mentioned above is that these will permit planning and maintenance and rehabilitation rather than doing as is done at present, that is, reacting to a present need to repair pavements in poor condition. Planning will also permit the use of more preventative maintenance which is considerably more cost-effective than the restorative maintenance that is done on a reaction basis. By being able to plan, the long-range cost of a maintenance and rehabilitation program can be reduced considerably, perhaps in some cases as much as 30 to 50 percent.

As a consequence, there is a need to begin now to develop a carefully structured data base composed of all available pavement information on a carefully selected group of airport pavements. The data include traffic and construction histories, thickness and materials properties of the layers, detailed climatic data, and periodic condition and deflection histories. It is from this limited data set that information will be drawn to improve and modify existing design and prediction equations. The number of sections should be kept as small as possible in order to reduce to a minimum the manpower that is required to acquire the periodic data. The set of airport pavements selected should include a range of traffic levels, pavement types, environmental conditions, and design thickness.

In addition to this limited data set, there is a need to compile pavement condition data on a considerably larger number of pavements so as to provide data for verifying the planning and design equations that are developed.

USE OF NON-DESTRUCTIVE TESTING

To determine the necessary parametes (e.g., material properties) for input into the improved mechanistic approach recommended above, we strongly urge the continued development of procedures for the nondestructive evaluation of existing pavements. These procedures should have the capability of providing measures of the stiffness characteristics of existing pavement systems including those of the components; indications of pavement characteristics in the vicinity of discontinuities (e.g., joint and cracks); and measures of environmental influences including moisture and temperature.

Consideration should be given to the magnitude of the load used to evaluate specific pavement cross sections to insure that nonlinear effects of load on material response and discontinuities will be adequately represented.

Investigations should not be limited to vibratory-type testing but should include other loading modes as well, such as those obtained from deflectometer-type equipment and impulse (falling weight) devices.

Analytical studies using finite element methodology and including the provision for dynamic effects should be utilized to insure that the various effects noted above are properly reflected in the nondestructive evaluations.

USE OF PROBABILISTIC CONCEPTS IN OVERLAY DESIGN

One of the crucial factors in the design of overlays is to determine the level of "reliability" that is required of the overlay. Reliability in an overlay primarily means that it will remain in service at least as long as was intended without requiring undue interruption of air traffic operations. As a consequence, the reliability levels that are required of low-volume general aviation airports are considerably lower than those of busy hub airports.

In technical terms, reliability is a probability of not failing or (1 - probability of failing to meet the established criteria). The factors that cause reliability to drop are large variabilities in materials properties, layer thicknesses, and projected air traffic volumes. Although little can be done to alter the variability in traffic volumes, there is much that can be done about the variability in materials properties and layer thicknesses, primarily through enhanced quality control in the construction operation.

However, even with the application of a rigorous program of quality control, there will always be an irreducible minimum amount of variability. The variability that is inherent in a pavement results in an uncertainty about the exact length of time or number of coverages that pavement can withstand. Knowing the mean and variance of the materials properties and layer thicknesses of a pavement permits the determination of an expected value (mean) and a variance of the life of an overlay. The required reliability of a pavement is found from the equation

$$\text{Expected life} - z(\text{variance of life})^{1/2} = \text{minimum required life}$$

The multiplier z is chosen from a table of constants which correspond with different levels of reliability. Example values of z and corresponding reliability levels are given below.

Reliability	Multiplier z
50	0.0
84	1.00
90	1.28
95	1.64
99	2.33
99.5	2.57
99.9	3.08

It is impossible to use reliability design unless information is known of the variability of the relevant materials properties, layer thicknesses, and projected future traffic. The advantage of using reliability design is that only those pavement thicknesses and quality control programs are used that will assure the required level of reliability. For any given level of variability of the design quantities (materials properties, thickness, traffic) a higher level of reliability will require a thicker and more costly design. The greater cost can usually be easily justified by the savings in down-time of the airport. If it cannot be justified in this manner, then the specified level of reliability is too high.

It is strongly recommended that among the data collected and stored in the long-term monitoring of pavements, the compilation of frequency distribution data on layer thickness, material properties, and traffic be included. It is further recommended as a long-range goal that all overlay design be done on a probabilistic basis as described in general above. It is also recommended that a study to determine the required level of reliability be undertaken based upon the cost savings to the user of decreased down-time of the airport facilities.

SUBSURFACE DRAINAGE

We strongly urge that guidelines be developed for the design of subsurface drainage systems that can be incorporated in existing pavements prior to overlaying or as part of the overlay structure. Developments should include: special drainage materials; procedures for estimating flow through partially saturated materials; special construction techniques that permit incorporation

of such drainage features in existing pavements; and procedures to insure effective maintenance of such facilities once installed.

PREDICTING MAINTENANCE AND REHABILITATION ACTIVITIES

One of the major benefits of developing predicting equations for airport pavements and overlays is to be able to take advantage of predicted life estimates to give lead time for planning, committing, and securing funds so that they will be available when they are needed to do preventive maintenance. This is much more cost effective than deferring maintenance until a later time when the pavement is in much worse condition and requires a more costly treatment. The deferral is usually due to a lack of available funds which could be at least partly averted by the use of advance planning.

A systematic procedure needs to be developed to use realistic predictive equations for pavements and overlays to make projections of funding needs, and to help in scheduling manpower, equipment, and materials for maintenance and rehabilitation activities. This procedure is expected to result in a considerable amount of money saved overall because it permits the use of more cost effective maintenance and rehabilitation methods and techniques.

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APPENDIX C: ANALYTICAL MODELS FOR PAVEMENT STRUCTURES

INTRODUCTION

To analytically compute stresses and deformations in concrete pavement structures, a number of different procedures are available. These procedures can generally be grouped into two categories, i.e., elastic plate on supporting foundation and the layered systems. The analysis of layered systems differs greatly both in assumptions and mathematical approaches from that of the plates on supporting foundation (either liquid foundation or elastic solids).

ELASTIC PLATE ON SUPPORTING FOUNDATION

This procedure is developed based on the classical theory of plates. The salient feature of the procedure is that it yields a two-dimensional solution and consequently simplifies the problem considerably when the finite element method is employed. Because the plates are thin and the deflection is small, the assumption that the deformations at the surface and at the bottom of the plate at a line normal to the plane of the plate are equal is qualified. Consequently, there is no variation of deformation in the direction of the thickness of the plate (linear variation of stress and strains), and the problem becomes a two-dimensional one.

ELASTIC PLATES ON DENSE-LIQUID FOUNDATION (WINKLER FOUNDATION)

The theory of plates on a continuous elastic foundation was first formulated by E. Winkler in 1867, who assumed the intensity of the reaction of the elastic foundation at any point was proportional to the deflection of the plate at that point. In other words, the settlement of the elastic foundation at any point on its surface was assumed to be proportional to the pressure between the plate and the foundation at the same point, and consequently to be independent of the pressure elsewhere. This corresponds physically to the problem of a plate on a liquid base. It is also necessary to assume that the reactive pressures are vertical only; frictional forces are developed, but they are neglected.

Closed-form solutions were developed for stress conditions in a concrete slab resting on a dense-liquid foundation by Westergaard[1-4*] and for

* References are found at the end of this Appendix.

temperature-induced stresses by Westergaard[5] and Bradbury.[6] Westergaard's formulas were then employed by Pickett and Ray[7] for developing influence charts, which have been used by the Portland Cement Association[8,9] for the design of highway and airport pavements. The H-51 computer program is used at WES for Westergaard-type solutions.[10] The discrete element computer programs (SLAB30 and SLAB49) developed at the University of Texas[11] in the late 1960's were based on the finite difference technique to analyze concrete slabs on a dense-liquid foundation. The method considers the slab to be an assemblage of elastic joints, rigid bars, and torsional bars. The means of modeling in this method proved helpful in visualizing the problem and forming the solution. The values given for pavement deflections are reasonable, but there are problems in achieving accurate stress values along the edges. In fact, serious problems exist in the analysis of joints, cracks, and gaps under the slab because of the nature of the method.

The finite-element computer programs developed based on the classical theory of plates (as used by the Westergaard solution) during the 1970's are essentially two-dimensional. The programs were developed at the USAE Construction Engineering Research Laboratory,[12,13] at the University of Kentucky,[14-17] at the University of Illinois (called ILLISLAB),[18] and at the U. S. Army Engineer Waterways Experiment Station (called WESLIQID and WESLAYER).[19] These programs yield results similar to those of the Westergaard's solution under similar conditions. The ILLISLAB and WESLIQID programs are essentially the same except for the difference in analyzing the load-transfer capability of the dowel bars across the joint. The salient features of the WESLIQID computer program which are not available in the Westergaard's solution are presented as follows.

MULTIPLE-WHEEL LOADS AND NUMBER OF SLABS

Any number of slabs (and any slab sizes) arranged in an arbitrary pattern can be handled in the program. Computer costs increase with increasing number of slabs. Multiple-wheel loads placed at any location on the slabs can be handled and the number of wheels is not limited.

SUBGRADE CONTACT OPTION

Complete subgrade contact condition was assumed in the Westergaard solution. The slab always has a full contact with the subgrade soil, and gaps are

not allowed between the slab and the subgrade no matter how much the slab has warped upward due to temperature change or the applying load. In other words, the slab is supported by a group of springs and the springs are always connected to the slab. In reality, the pavement can lose subgrade support at some part due to temperature warping, pumping, and plastic deformation of the subgrade.

An iterative scheme is developed in the computer program to determine the subgrade contact condition. A node that is not in contact before loading may become in contact after loading and vice versa. For pavements, if certain nodal points are known to be not in contact with the subgrade due to pumping, the amount of gaps at those nodes can be specified and the reactive forces at those nodes are deleted in the computations. If certain nodal points near pavement edges and corners are first assumed to be in full contact but lose contact later due to temperature warping, the stresses in the concrete slab are computed based on the subgrade support conditions determined at the last cycle of the iterative process.

TWO-LAYER SLABS

The program can be applied to two-layer slabs, either bonded or unbonded. The advantage of this option is that concrete pavements with cement-stabilized or lean concrete base and overlay can be analyzed. Although the program considers a two-layer slab, the assumption that the deformation at the top and at the bottom of the two slabs are equal still holds. The assumption is obviously a valid one. The procedure used in the WESLIQID to analyze the stress condition in a two-layer slab when there is no bond between the layers is described as follows.

a. Slab deflections $[\delta]$ are computed based on the total stiffness matrix $[K]$ and external forces $[F]$, i.e., $[F] = [K][\delta]$. The total stiffness matrix is the sum of the stiffness matrix of each layer and that of the subgrade soil.

b. The moment $[M_i]$ at each node of each slab is computed using the equation $[M_i] = \frac{1}{4ab}[K_i][\delta]$, where $[K_i]$ is the stiffness matrix of the i^{th} layer and $i = 1$ or 2; a and b are dimensions of the element.

c. The stress σ at each node is computed from the computed moment through the equation $\sigma = \frac{MC}{I}$.

VARIABLE THICKNESS OF CONCRETE SLABS

This option in useful for pavements with thickened edge joints or pavements adjacent to a cement-stabilized shoulder. This option may also be useful in the following special cases:

a. Overlay concrete pavements when a crack exists in the base concrete slab. The thickness in the particular element along the crack in the base concrete slab can be input as zero. If the element width is limited, a crack condition can be assumed for these elements.

b. Concrete slabs on grade with T-sections. The slabs can be analyzed using the option of two-layer slabs assuming the bonded condition for the interface. In areas where a T-section does not exist, the thickness of the base layer can be input as zero.

c. Slab shape other than rectangular. The program (as is true for most programs) is designed for rectangular slabs only. For slab shapes other than rectangular, such as hexagonal, the program can circumvent the limitation by assigning the slab thickness as zero in those undesirable nodes.

VARIABLE MODULUS OF SUBGRADE SUPPORT

One salient feature of this option is the capability of analyzing the handling stresses in a precast concrete slab. An extremely large subgrade modulus k value, such as 1,000,000 pci, can be assigned at the hanging points, while the k values at all the other nodes are assumed to be zero.

STRESSES, STRAINS, AND DEFLECTIONS IN THE SUPPORTING SUBGRADE SOIL

Once the subgrade reactive forces between the subgrade and the slab at each node are determined, stresses and strains in the supporting subgrade soil can be computed. The stresses and strains are induced by the nodal reactive forces, but the forces are acting in the direction opposite to those when the stress conditions in the slab were computed. When the subgrade soil is represented by the Winkler foundation (WESLIQID program), the Boussinesq's equations can be used to compute the stresses and strains induced by the concentrated nodal forces. In order to use the equations, an equivalent elastic modulus E corresponding to the modulus of subgrade reaction k (used in the program to compute stress condition in the slab) should be selected. Because the stresses and strains in the subgrade soil under the concrete slabs are very small, the principle of superposition is valid and is used to compute the

stresses and strains in the soil induced by all the nodal forces.

LOAD TRANSFER ACROSS JOINTS AND CRACKS

The program provides three options for specifying shear transfer but only one for moment transfer. The three options for shear transfer are (a) efficiency of shear transfer, (b) spring constant, and (c) diameter and spacing of dowels. The only option for moment transfer is to assume an efficiency of moment transfer across the joint.

ELASTIC PLATES ON ELASTIC FOUNDATIONS

The theory of plates on elastic solids was developed assuming the foundation has the properties of a semi-infinite elastic body; i.e., the intensity of the reaction or settlement of the elastic foundation at any point is not independent of the settlement elsewhere. Solutions to this type of problem are available.[7,16,19,20,21] The WESLAYER program[19] is essentially similar to WESLIQID, except that the dense-liquid foundation is replaced by the elastic layered system.

LAYERED SYSTEMS

This category can be divided into three subcategories. They are (a) three-dimensional solids with two-dimensional simplification, (b) true three-dimensional finite element models, and (c) multilayer elastic systems (Burmister solution). These models are discussed as follows:

THREE-DIMENSIONAL SOLIDS WITH TWO-DIMENSIONAL SIMPLIFICATION

Duncan, Monismith, and Wilson[22] developed an axisymmetric finite element program to solve stress conditions in primarily flexible-type pavements. Its limitation is that only the interior loading condition is applicable. The plane strain finite-element formulations[23,24] can be made applicable to determine interior and edge stresses but cannot evaluate corner stresses. An indirect procedure was used[25,26] in that the interior strip load required to give the same interior stress, as computed by axisymmetric or elastic-layer solutions, is determined by trial. This equivalent strip load is then used to calculate edge stress. The prismatic space finite-element method[27] was designed to model three-dimensional pavement problems, but is essentially two-dimensional, with the third dimension introduced into the idealization by

expressing the load as a Fourier series in this direction. This configuration permits determination of edge but not corner stresses.

TRUE THREE-DIMENSIONAL FINITE ELEMENT MODELS

The most general method available is the eight-node brick three-dimensional program. This is a three-dimensional version of Solid SAP.[23] Although this program is considered to be the most appropriate to model a pavement, the time and computer memory required can hinder the use of the program, at least on a regular basis.

MULTILAYER ELASTIC SYSTEMS (BURMISTER SOLUTION)

The elastic solution for two- and three-layer systems was first developed by Burmister[28,29] and later extended by Mehta and Veletsos[30] to multilayered systems. Although the method was developed for a three-dimensional problem, it is essentially two-dimensional because of the restriction of axial symmetry. For multiple-wheel problems, the method of superposition is used. The solution of the problem is based on the theory of elasticity. The material in each layer is assumed to be weightless, homogeneous, isotropic, and linearly elastic. The lowermost layer is considered to be of infinite extent in both the horizontal and the vertical directions. A continuous surface of contact between layers is assumed, and the interfaces are considered to be either rough or smooth. Across a rough interface there is no relative displacement in the horizontal direction, and the shearing stress is continuous. At a smooth interface, there is no shearing stress, and the radial displacements on either side of the common surface of contact are generally different.

Several computer programs have been developed based on the multilayer elastic theory to solve stress conditions in pavements. The most commonly used ones are CHEVRON[31] and BISAR.[32] The former is limited to a single-wheel load and the latter can be used for multiple-wheel loads. The CHEVRON [31] program was later extended by Chou[33] and Ahlborn[34] to account for the effect of the nonlinear properties of pavement materials on pavement responses. The BISAR[32] program was also adopted by the U. S. Army Corps of Engineers for the design of rigid pavements.[35]

The disadvantage of using the multilayer elastic theory for rigid pavement design is that the slab is assumed to be finite in extent in the

horizontal plane, and consequently only the interior load case can be analyzed. Corner and edge stresses and joint conditions cannot be analyzed. For overlay design the BISAR program can assume the interface condition to be either smooth (unbonded) or rough (bonded); the program also has the capability of analyzing conditions in between.

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APPENDIX D: DERIVATION OF FORMULA FOR PREDICTING SUBGRADE k-VALUE FROM DEFLECTIONS

The finite element computer program WESLIQID[31]* was used to compute the deflections at the center of concrete slabs subjected to a circular load that is placed at the center of a square slab. Computations were made for varying pavement properties shown in Table D-1. Correlations between the deflection w and subgrade modulus k were established based on the computed results.

Table D-1

Pavement Properties Used in Computations

Load, P, lb	14,000
Diameter of the loaded area, D, in.	6, 12, 24
Slab dimension, L, ft	10, 15, 20, 25
Slab thickness, h, in.	6, 10, 15, 20
Concrete modulus, E, psi	2,000,000, 3,000,000, 4,000,000, 5,000,000
Concrete Poisson's ratio, ν	0, 15
Subgrade modulus, k, pci	25, 50, 75, 100, 200, 500
Temperature differential, t*	-2, -1, 0, 0.75, 1.5, 3

* Temperature differential is defined to be the Fahrenheit degree per inch of the concrete slab. Positive differential indicates the temperature at the slab surface is colder than that at the slab bottom; the reverse is true for negative differential.

The magnitude of the load was held constant in the computation because the computer program assumes the validity of linear elasticity, i.e., the deflection is linearly proportional to the load. The computed results indicate that the deflections are rather insensitive to the loaded area for the range used in the computation. This is verified by the computed deflections shown in Table D-2 for a 15 ft by 15 ft slab.

Table D-3 shows the computed deflections for a 15 by 15 ft concrete slab for various slab thicknesses, t ; concrete moduli, E ; and subgrade moduli,

* The references referred to by superscript numerals in this Appendix are found in the References section at the end of the main text.

Table D-2

Computed Deflections for Three Diameters of Loaded Area, Zero Temperature Gradient

h in.	E psi	D in.	Deflections, in. k 25 pci	k 75 pci	k 200 pci	k 500 pci
10	3,000,000	6	0.02778	0.01414	0.00811	0.00485
10	3,000,000	12	0.02761	0.01399	0.00797	0.00473
10	3,000,000	24	0.02729	0.01370	0.00772	0.00451
20	3,000,000	6	0.01878	0.00720	0.00347	0.00196
20	3,000,000	12	0.01875	0.00718	0.00345	0.00194
20	3,000,000	24	0.01871	0.00713	0.00341	0.00191
10	4,000,000	6	0.02543	0.01253	0.00715	0.00427
10	4,000,000	12	0.00530	0.01242	0.00705	0.00418
10	4,000,000	24	0.02506	0.01219	0.00685	0.00400

Table D-3

Computed Deflections for a 15 ft by 15 ft Square Concrete Slab Under Various Conditions, 14,000-lb Circular Load, 12-in Diameter, Zero Temperature Gradient

t, in.	E, psi	Deflection, in., for Values of k in pci 25	50	75	100	200	500
6	2,000,000	0.06004	0.04040	0.03212	0.02735	0.01871	0.01140
10	2,000,000	0.03180	0.02098	0.01662	0.01410	0.00949	0.00566
15	2,000,000	0.02226	0.01330	0.01015	0.00848	0.00863	0.00334
20	2,000,000	0.01947	0.01076	0.00782	0.00633	0.00397	0.00231
6	3,000,000	0.05046	0.03395	0.02693	0.02288	0.01556	0.00946
10	3,000,000	0.02761	0.01776	0.01399	0.01185	0.00797	0.00473
15	3,000,000	0.02068	0.01188	0.00887	0.00731	0.00476	0.00280
20	3,000,000	0.01875	0.01009	0.00718	0.00571	0.00345	0.00194
6	4,000,000	0.04460	0.03002	0.02381	0.02020	0.01368	0.00829
10	4,000,000	0.02530	0.01590	0.01242	0.01049	0.00705	0.00418
15	4,000,000	0.01986	0.01113	0.00817	0.00665	0.00424	0.00248
20	4,000,000	0.01839	0.00973	0.00684	0.00538	0.00316	0.00173
6	5,000,000	0.04063	0.02728	0.02164	0.01835	0.01239	0.00748
10	5,000,000	0.02384	0.01467	0.01136	0.00956	0.00641	0.00379
15	5,000,000	0.01936	0.01066	0.00772	0.00623	0.00390	0.00225
20	5,000,000	0.01817	0.00952	0.00663	0.00518	0.00298	0.00159

k . The temperature gradient in the concrete slab is assumed to be zero. The results are plotted in Figure D-1 for k-values of 50, 100, 200, and 500 pci. Figure D-1 shows that for a given pavement thickness, h , the relationships between the deflection, w , and the concrete modulus, E , are linear on the logarithmic scale for a given subgrade modulus, k , and the straight lines so drawn are approximately parallel to one another. For a given E value, the relationship between the deflection w and the subgrade k-value is also linear on the logarithmic scale, which is plotted in Figure D-2 for concrete modulus $E = 5,000,000$ psi . Although the relationships are linear, the slopes of the lines vary with the slab thickness h . Although the relationships shown in Figure D-2 are for the case of one E modulus, similar relationships can also be obtained for other E moduli.

To determine the subgrade modulus k from the measured deflection, the relationships shown in Figure D-1 can be used. Attempts were made to formulate expressions based on the data tabulated in Table D-3. Mathematical expressions can be written either based on equations of the straight lines that are drawn through the data points shown in Figure D-1 or using the least-square technique based on the data in Table D-3. Both methods were employed and the derived expressions were very similar. Results obtained from the least-squares technique are presented as follows.

The Biomedical Computer Program (BMD)[38] for regression analysis was used. The program uses the stepwise regression procedure employing the least-squares methodology to select the best model to fit the data. The procedure starts with the simple correlation matrix and enters into regression the independent variable most highly correlated with the dependent variable. Using partial correlation coefficients, it then selects as the next variable to enter regression, that independent variable whose partial correlation with the dependent variable is highest and so on. The procedure reexamines at every stage of the regression the variables incorporated into the model in previous stages. The procedure does this by testing every variable at each stage as if it were entered last, and checks its contribution by means of the partial F test. The process is stopped when essentially no additional variable significantly improves the precision of the model.

Because logarithmic relations are evident in the straight-line presentations shown in Figure D-1, i.e., logarithmic relationships exist among the variables, the variables E , h , k , and w were transformed to logarithmic

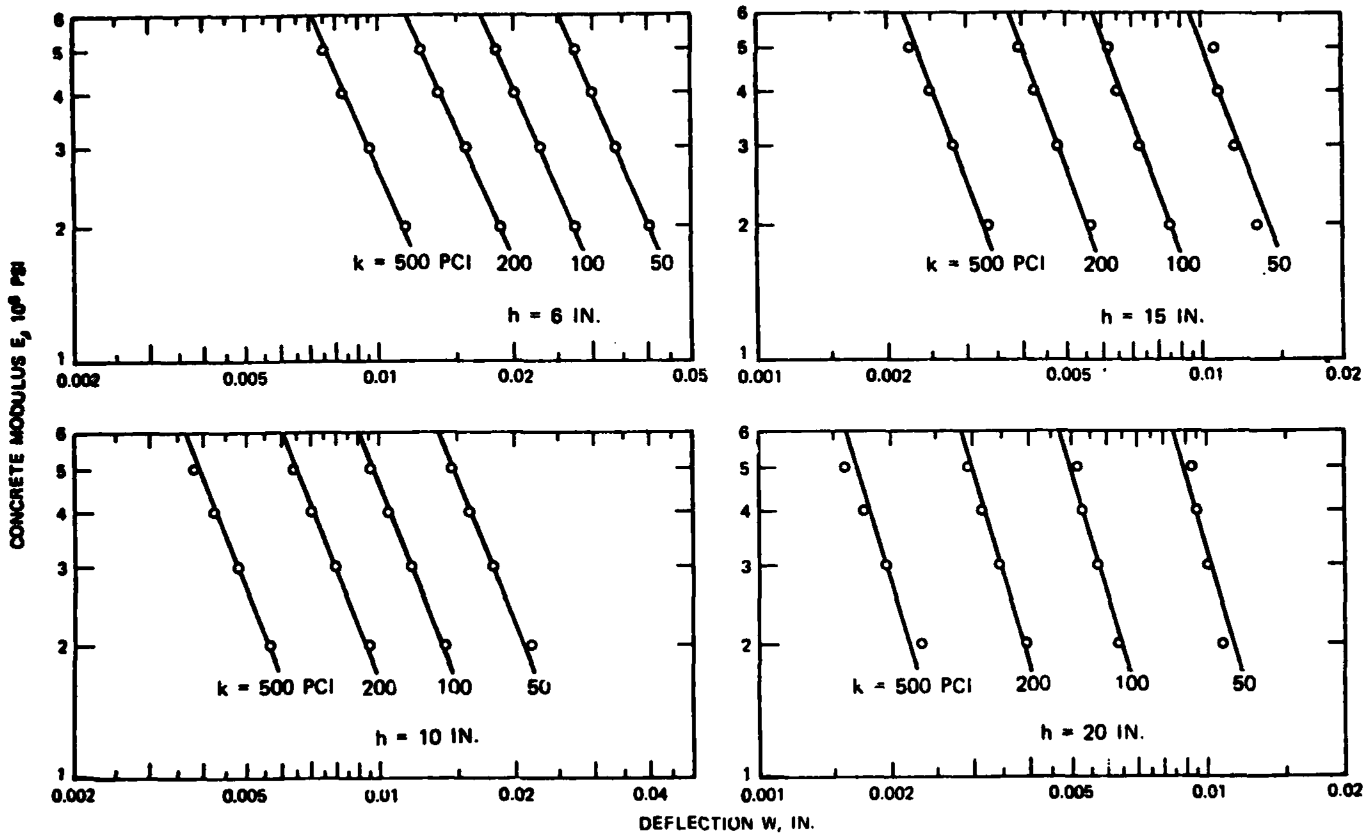

Figure D-1. Relationships between deflections and other pavement properties

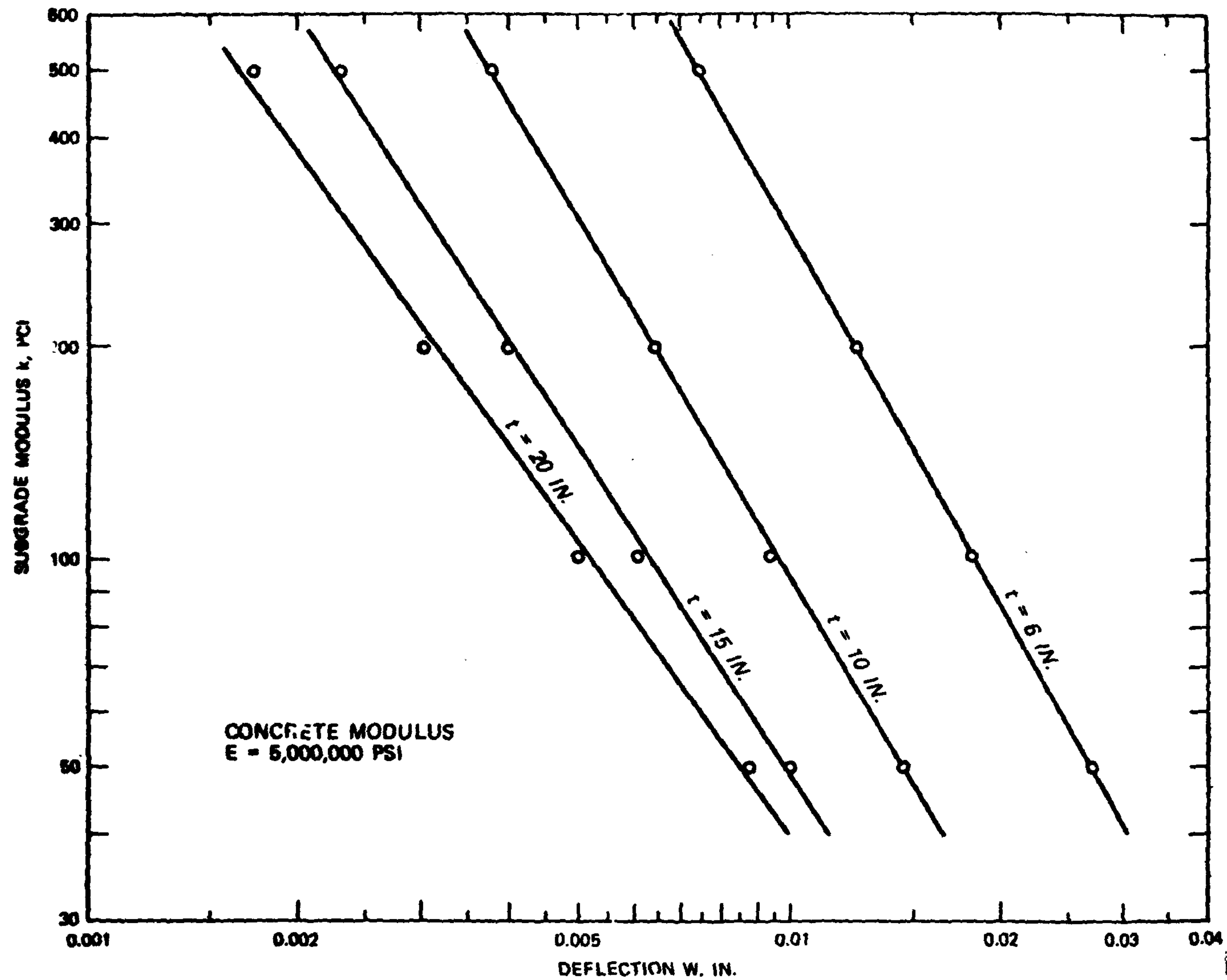

Figure D-2. Logarithmic relationships between deflections and subgrade modulus

form prior to the application of the regression analysis to insure better correlations. The mathematical equations for the four slab thicknesses are shown as follows

$$k = \frac{10294}{w^{1.779}\ E^{0.7619}} \quad \text{for } h = 6 \text{ in.}$$

$$k = \frac{1302}{w^{1.71497}\ E^{0.68427}} \quad \text{for } h = 10 \text{ in.}$$

$$k = \frac{246}{w^{1.5415}\ E^{0.5642}} \quad \text{for } h = 15 \text{ in.}$$

$$k = \frac{86.5}{w^{1.40465}\ E^{0.4738}} \quad \text{for } h = 20 \text{ in.} \qquad \text{(D-1)}$$

The general expression for Equation D-1 can be written as

$$k = \frac{\alpha}{w^{\beta}\ E^{\gamma}} \quad \text{for a given } h \qquad \text{(D-2)}$$

To incorporate the quantitative values of the slab thickness h in the equation, it can be noted that the values of α , β , and γ in Equation D-1 decrease with increasing h . Figure D-3 shows the relationship between the constant α and the slab thickness h which is linear on the logarithmic scale. Using the least-squares technique, the mathematical equation has the form

$$\alpha = \frac{13,342,400}{h^{3.98678}} \qquad \text{(D-3)}$$

Unlike the other relationships, the relationship between the constant β and the slab thickness h and the relationship between the constant γ and the slab thickness h are linear only on the semilogarithmic scale. The relationships are shown in Figure D-4. The resulting equations have the form

$$\beta = \log_{10}^{-1} (0.29417 - 0.00733h) \qquad \text{(D-4)}$$

$$\gamma = \log_{10}^{-1} (-0.029657 - 0.014737h) \qquad \text{(D-5)}$$

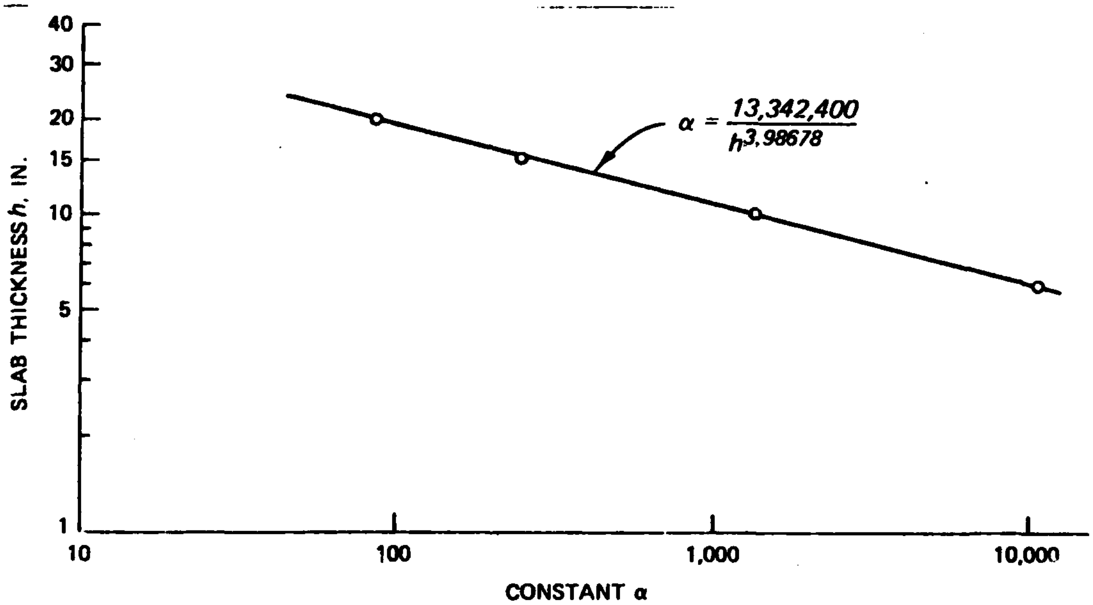

Figure D-3. Relationship between the constant, α , and slab thickness, h

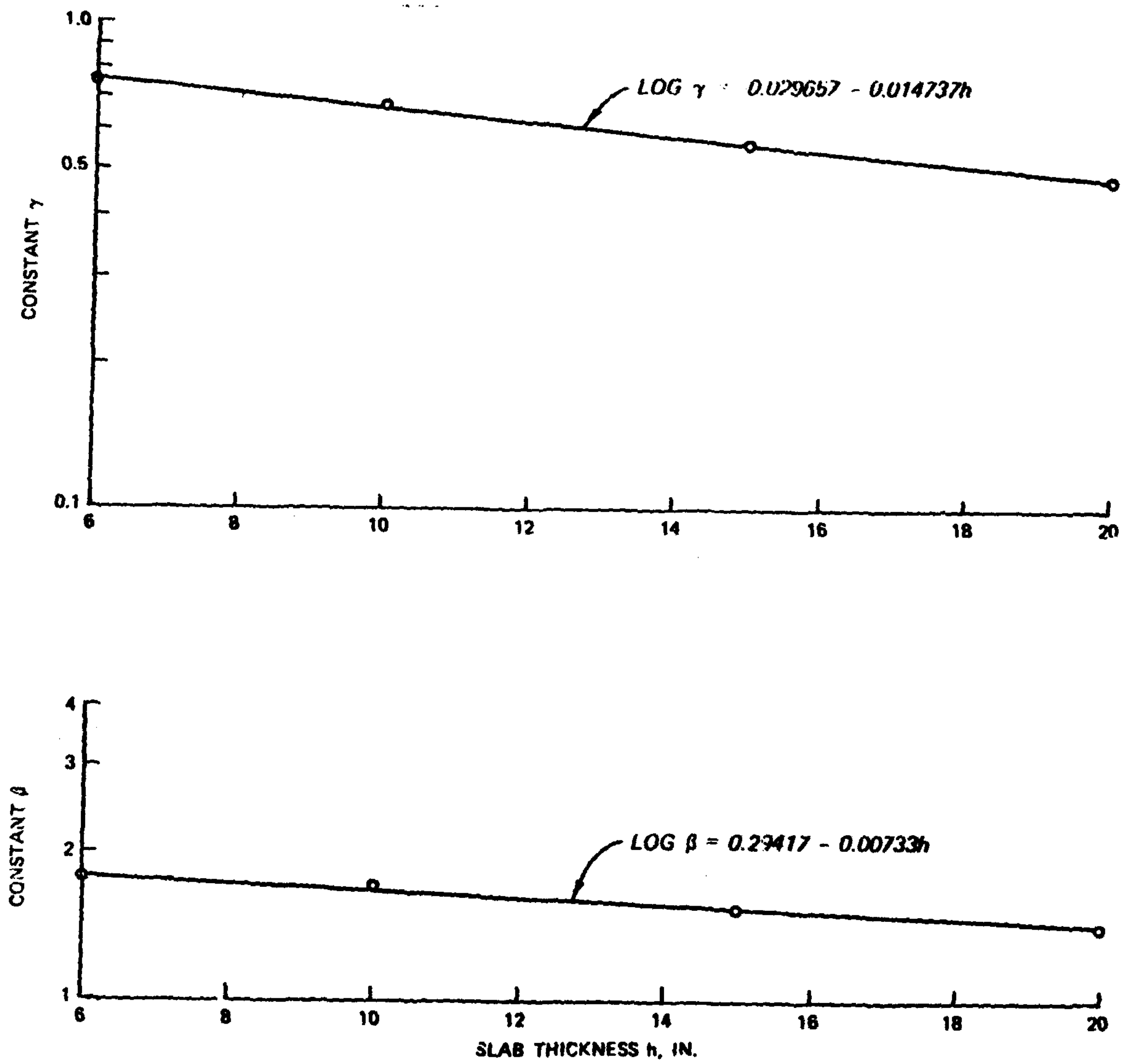

Figure D-4. Relationships between constants β and γ and slab thickness, h

Substituting Equations D-3 to D-5 into Equation D-2 yields

$$k = \frac{13,342,400}{h^{4.0} \times w^{\log^{-1}(0.29417-0.00733h)} \times E^{\log^{-1}(-0.029657-0.014737h)}}$$

or

$$k = \frac{13,342,400}{h^{4.0} \times w^{\exp(2.3A)} E^{\exp(2.3B)}} \tag{D-6}$$

where

$$A = 0.29417-0.00733h$$

$$B = -0.0297-0.01474h$$

$$\exp(2.3A) = e^{2.3A}$$

Equation D-6 can be used to determine the subgrade k-value based on the measured slab deflection with the known slab thickness and estimated concrete modulus, E . The magnitude of the load is 14,000 lb and the diameter of the loaded area is 12 in. The temperature gradient across the concrete slab is assumed to be zero; i.e., the concrete slab does not warp due to temperature change.

It should be pointed out that while Equation D-6 was formulated using the regression analysis procedure, the selections of the variable forms at different stages were assisted by visual examination of the relationships plotted in Figure D-1 to D-4. Two other regression models were formulated using the same computer program based on the same data shown in Table D-1. The two models have the forms

$$\log_e k = f(\log_e E, \log_e w, \log_e h) \tag{D-7}$$

$$\log_e k = f(h, \log_e h, h^2, \frac{1}{h}, E, \log_e E, E^2, \frac{1}{E}, w, \log_e w, w^2, \frac{1}{w},$$

$$\log_e h\ h\log_e, \log_e h \log_e w, \log_e E \log_e w) \tag{D-8}$$

Equation D-7 assumes that the relationships among the variables are independent. The additional terms in Equation D-8 recognize the effects of the interaction of independent variables on the dependent variable. For example, the addition

of the term $\log_e E \log_e w$ assumes that the effect of subgrade, k, on slab deflection, w, is affected by the degree of stiffness of the concrete slab, E. The derived equations corresponding to Equations D-7 and D-8 are, respectively

$$\log_e k^* = 10.678 - 1.57 \log_e w - 0.583 \log_e E - 1.84 \log_e h \qquad \text{(D-9)}$$

where

Multiple correlation coefficient (R^2) = 0.9897

Standard error of estimate (SEE) = 0.1259

$$\begin{aligned}\log_e k = & -5.497 + 0.165h + 55.74w - 451.8w^2 + 0.733/\bar{E} \\ & -1.22 \log_e h - 0.739 \log_e \bar{E} - 4.075 \log_e w \\ & + 0.644 \log_e h \log_e \bar{E} + 0.711 \log_e \bar{E} \log_e w \\ & + 0.252 \log_e \bar{E} \log_e w \end{aligned} \qquad \text{(D-10)**}$$

where

$R^2 = 0.9988$

SEE = 0.01302

The multiple correlation coefficient R^2 and the standard error of the estimate SEE were computed by the following equations, respectively:

$$R^2 = \frac{\sum(\log_e k - \log_e \bar{k})^2 - \sum(\log_e k - \log_e \hat{k})^2}{\sum(\log_e k - \log_e \bar{k})^2} \qquad \text{(D-11)}$$

$$SEE = \sqrt{\frac{\sum(\log_e k - \log_e \hat{k})^2}{N - 2}} \qquad \text{(D-12)}$$

where $\hat{k}$ is the predicted k value for the regression model, $\bar{k}$ is the mean of all k-values, and N is the total number of data points. The R^2 represents the proportion of the total variation about the mean, $\log_e \bar{k}$, explained by the regression model. The closer R^2 is to 1.0, the closer the data cases

---

* Equation D-9 can be converted into

$$k = \frac{43,373}{w^{1.57} E^{0.583} h^{1.84}}$$

** $\bar{E}$ = 1,000,000 E

lie on the predicted line, and the closer to 0.0, the greater the scatter of data about the line. In the case of Equation D-9, $R^2 = 0.9897$ indicates that the regression equation obtained explains 98.97 percent of the total variation. The magnitude of the SEE is simply the standard deviation of the residuals, and may be interpreted approximately as the average error in predicting $\log_e k$ from the regression model. The size of the SEE can be compared with the mean of all $\log_e k$'s. Ideally, the SEE should be much smaller than the mean of all $\log_e k$'s.

It should be pointed out that since the dependent variable k was transformed into the logarithmic form in Equations D-9 and D-10, the derived expressions for R^2 and SEE are in terms of the logarithmic form of k-values. Unfortunately, the physical meaning of R^2 and SEE in the logarithmic form is vague, and the accuracy of the regression models can only be determined by comparisons between the actual k-values and the predicted ones. Such a comparison was made among Equations D-6, D-9, and D-10 and the results are presented in Table D-4.

The means and the standard deviations for the computed percent differences were computed and presented in Table D-4. The larger the value of the standard deviation, the greater the variation of the computed percent differences. The SEE between the actual and the predicted k-values were also computed for the three equations and are shown in the table.

The results presented in Table D-4 indicate that Equation D-10 yields the best results. This is understandable because more terms are included in Equation D-10. The first five terms in Equation D-10 assume a semilogarithmic relationship between the independent variables, W , E , and h and the dependent variable k . The next three terms take into account the logarithmic nature of the relationship, and the last three terms account for the effects of interaction among the independent variables. Although both Equations D-6 and D-9 have three terms, predictions yielded by Equation D-6 are much better than those given by Equation D-9. The logarithmic relationships were assumed and fixed between the dependent variable and the independent ones in formulating Equation D-9. In Equation D-6, however, the logarithmic relationship was first assumed in formulating Equation D-1 based on the plotted logarithmic results shown in Figures D-1 and D-2, but semilogarithmic relationships were introduced later in formulating the final equation (D-6) because of the semilogarithmic relationship plotted in Figure D-4. The

Table D-4

Comparisons of the Three Regression Equations

Deflection in.	Slab Thickness, h, in.	Concrete Modulus, E psi	Actual Subgrade Modulus, k, pci	Equation D-6 Values k	Equation D-6 Values Percent Difference	Equation D-9 Values k	Equation D-9 Values Percent Difference	Equation D-10 Values k	Equation D-10 Values Percent Difference
0.04040	6	2,000,000	50	50.26	0.52	52.16	4.33	49.57	-0.84
0.02735	6	2,000,000	100	100.61	0.61	96.24	-3.75	99.49	-0.50
0.01871	6	2,000,000	200	197.69	-1.15	174.68	-12.65	199.43	-0.28
0.01140	6	2,000,000	500	477.29	-4.54	380.24	-23.95	538.44	7.68
0.03395	6	3,000,000	50	50.29	0.57	54.10	8.21	50.29	0.58
0.02288	6	3,000,000	100	101.47	1.47	100.53	0.53	97.63	-2.36
0.01556	6	3,000,000	200	201.49	0.74	184.16	-7.91	195.10	-2.44
0.00946	6	3,000,000	500	488.36	-2.32	402.27	-19.54	522.48	4.49
0.03002	6	4,000,000	50	50.27	0.54	55.49	10.98	50.75	1.51
0.02020	6	4,000,000	100	101.72	1.72	103.36	3.36	96.91	-3.08
0.01368	6	4,000,000	200	203.49	1.74	190.60	-4.69	193.59	-3.20
0.00829	6	4,000,000	500	496.08	-0.78	418.45	-16.30	516.38	3.27
0.02728	6	5,000,000	50	50.28	0.56	56.61	13.23	51.07	2.15
0.01835	6	5,000,000	100	101.81	1.81	105.51	5.51	96.49	-3.50
0.01239	6	5,000,000	200	204.76	2.37	195.48	-2.25	192.65	-3.67
0.00748	6	5,000,000	500	502.52	0.50	431.73	-13.65	512.95	2.59
0.02098	10	2,000,000	50	54.64	9.27	57.01	14.02	51.51	3.03
0.01410	10	2,000,000	100	105.80	5.80	106.39	6.39	96.23	-3.76
0.00949	10	2,000,000	200	204.38	2.19	198.10	-0.94	191.53	-4.23
0.00566	10	2,000,000	500	482.71	-3.45	445.94	-10.81	511.55	2.31
0.01776	10	3,000,000	50	55.04	10.08	58.45	16.91	52.63	5.27
0.01185	10	3,000,000	100	107.87	7.87	110.33	10.33	98.24	-1.75
0.00797	10	3,000,000	200	208.62	4.31	205.67	2.83	193.08	-3.45
0.00473	10	3,000,000	500	496.79	-0.64	466.58	-6.68	507.06	1.41
0.01590	10	4,000,000	50	54.64	9.27	58.80	17.60	53.50	7.01
0.01049	10	4,000,000	100	109.10	9.10	112.96	12.96	100.62	0.62
0.00705	10	4,000,000	200	211.27	5.63	210.82	5.41	195.75	-2.12
0.00418	10	4,000,000	500	503.89	0.77	478.99	-4.20	504.28	0.85
0.01467	10	5,000,000	50	53.85	7.69	58.57	17.15	53.96	7.92
0.00956	10	5,000,000	100	109.75	9.75	114.74	14.74	102.48	2.48
0.00641	10	5,000,000	200	213.35	6.67	214.91	7.45	198.18	-0.90
0.00379	10	5,000,000	500	511.21	2.24	490.42	-1.91	504.30	0.86
0.01330	15	2,000,000	50	58.38	16.76	55.30	10.60	50.92	1.84
0.00848	15	2,000,000	100	116.15	16.14	112.10	12.10	99.27	-0.72
0.00563	15	2,000,000	200	217.21	8.60	213.25	6.62	193.65	-3.17
0.00334	15	2,000,000	500	482.45	-3.51	484.07	-3.18	482.45	-3.50
0.01188	15	3,000,000	50	55.25	10.50	52.11	4.23	50.53	1.06
0.00731	15	3,000,000	100	116.06	16.06	111.71	11.71	101.18	1.18
0.00476	15	3,000,000	200	223.58	11.79	219.08	9.54	198.76	-0.61
0.00280	15	3,000,000	500	503.08	0.61	503.98	0.79	484.57	-3.08
0.01113	15	4,000,000	50	51.94	3.88	48.81	-2.36	50.03	0.06
0.00665	15	4,000,000	100	114.12	14.11	109.58	9.58	102.11	2.11
0.00424	15	4,000,000	200	227.03	13.51	222.12	11.06	203.13	1.56
0.00248	15	4,000,000	500	515.29	3.05	515.56	3.11	486.17	-2.76
0.01066	15	5,000,000	50	48.95	-2.10	45.86	-8.27	49.45	-1.09
0.00623	15	5,000,000	100	111.24	11.23	106.58	6.58	101.96	1.96
0.00390	15	5,000,000	200	227.59	13.79	222.36	11.18	204.86	2.43
0.00225	15	5,000,000	500	527.53	5.50	527.35	5.47	489.90	-2.01
0.01076	20	2,000,000	50	52.17	4.34	45.43	-9.13	49.91	-0.16
0.00633	20	2,000,000	100	109.93	9.92	104.49	4.49	103.21	3.21
0.00397	20	2,000,000	200	211.69	5.84	217.37	8.68	208.97	4.48
0.00231	20	2,000,000	500	452.95	-9.41	508.66	1.73	499.23	-0.15
0.01009	20	3,000,000	50	47.12	-5.75	39.67	-20.65	48.61	-2.77
0.00571	20	3,000,000	100	104.85	4.84	96.97	-3.02	101.57	1.57
0.00345	20	3,000,000	200	212.77	6.38	213.89	6.94	209.48	4.74
0.00194	20	3,000,000	500	477.64	-4.47	528.10	5.62	504.98	0.99
0.00973	20	4,000,000	50	43.27	-13.45	35.50	-28.98	48.09	-3.81
0.00538	20	4,000,000	100	99.47	-0.53	90.02	-9.97	100.07	0.07
0.00316	20	4,000,000	200	210.03	5.01	207.57	3.78	208.46	4.23
0.00173	20	4,000,000	500	489.55	-2.09	534.50	6.90	505.00	1.00
0.00952	20	5,000,000	50	40.14	-19.71	32.26	-35.47	47.71	-4.56
0.00518	20	5,000,000	100	94.38	-5.62	83.87	-16.12	98.40	-1.59
0.00298	20	5,000,000	200	205.18	2.59	199.81	-0.09	205.56	2.78
0.00159	20	5,000,000	500	495.86	-0.82	535.73	7.14	501.80	0.32
Mean					3.25		0.83		0.37
R^2					6.88		11.46		3.00
SEE				12.77		28.48		8.30	

inclusion of the semilogarithmic relationship has greatly improved the accuracy of Equation D-6.

It should be pointed out that the SEE of 8.3 computed for Equation D-10 shown in Table D-4 was based on the real k-values and the SEE of 0.01302 in Equation D-10 was obtained based on the logarithmic form of the k-values.

Equations D-6, D-9, and D-10 were developed for a constant load of 14,000 lb. While the deflection is linearly proportional to the applied load because of the assumption of linear elasticity, the relationship between the subgrade k-value and the load is not linear and can be determined from the relationship in Equation D-6. Equation D-6 can be rewritten as

$$w = \left[\frac{13,342,400}{h^{4.0}\ k\ E^{\exp(2.3B)}}\right]^{\frac{1}{\exp(2.3A)}}$$

When the load P is considered, the equation becomes

$$w = \frac{P}{14,000}\left[\frac{13,342,400}{h^{4.0}\ k\ E^{\exp(2.3B)}}\right]^{\frac{1}{\exp(2.3A)}} \qquad \text{(D-13)}$$

Rewriting the equation in terms of k , it becomes

$$k\left(\frac{14,000}{P}\right)^{\exp(2.3A)} = \frac{13,342,400}{h^{4.0}\ w^{\exp(2.3A)}\ E^{\exp(2.3B)}}$$

or

$$k = \frac{13,342,400}{h^{4.0}\ w^{\exp(2.3A)}\ E^{\exp(2.3B)}}\left(\frac{P}{14,000}\right)^{\exp(2.3A)} \qquad \text{(D-14)}$$

where A and B are as previously defined. The last term in the equation is the multiplying factor for adjusting the effect of the load on the computed k-values.

Equations D-6, D-9, and D-10 compute the subgrade k-values from the input concrete modulus E , deflection w , and slab thickness h . The equations are valid only when the slab size is near 15-ft square and the

temperature differential is zero. The considerations of temperature and slab size effects in the predicting equations are discussed in the following paragraphs.

The WESLIQID computer program was again used to compute deflections in a concrete slab of varying thickness. Temperature differentials across the slab thickness were considered. The temperature differential t ranges from 0 to +3 and from 0 to −2 (see Table D-1). Constant values of concrete modulus $E = 4{,}000{,}000$ psi, slab size $L = 15$ ft, and load $P = 14{,}000$ lb were used in the computation. The stepwise regression analysis was used to derive the best-fit equations. The derived regression equations and the R^2 and SEE values in terms of logarithmic k-values are represented as follows:

a. For positive temperature differentials (nighttime condition):

$$\log_e k = -0.531 - 2.058 \log_e h + 6.078/h - 1.948 \log_e w + (1.36h + 0.354h^2)w - 10.162h^2w^2 - (0.748 - 0.0011h - 15w + 0.133 \log_e w + 219.2w^2)h^2 \qquad \text{(D-15)}$$

$$R^2 = 0.9987$$

$$SEE = 0.0482$$

b. For negative temperature differentials (daytime condition):

$$\log_e k = 7.739 - 38.99wh + h^2(0.00015 \log_e w + 101.9w^2) - t^2 (7.76w - 0.0277h - 486.5w^2 + 11.9h^2w^2) \qquad \text{(D-16)}$$

$$R^2 = 0.9558$$

$$SEE = 0.2649$$

It is seen that the correlation obtained for Equation D-16 for a negative temperature differential is not as good as that for Equation D-15 for the case of a positive temperature differential.

The WESLIQID computer program was also used to compute deflections in a concrete slab of varying slab size L. Computations were made based on the following assumptio : (a) temperature differential was zero, (b) load P was 14,000 lb, and (c) the concrete modulus E was 4,000,000 psi. The derived regression equation has the form:

$$\log_e k = -1.66 + 0.5234h - 0.0071L + 6.398/L + \log_e w ($$
$$-2.0636 + 0.506 \log_e h - 0.049L + 0.00079L^2$$
$$-0.00092hL) - \log_e L\ (0.1738h + 0.288 \log_e h) \qquad \text{(D-17)}$$

where R^2 is 0.9984 and SEE is 0.0529.

To derive regression models for subgrade modulus E that account for the effects of all the variables, i.e., concrete modulus E , slab thickness h , slab size L , and temperature differential t , separate computations can be made to compute the deflections for each case and a stepwise regression analysis can be made for all the data points computed. However, the number of computations when such a procedure is used would be formidably large and the work involved would be very cumbersome. Attempts were thus made to combine the regression models illustrated in Equations D-6, D-9, D-10, D-15, D-16, and D-17. Since Equations D-15 and D-16 for considering temperature effects are rather lengthy, it is more convenient to estimate the modifying factors for the concrete modulus E from Equations D-6, D-9, and D-10 for the slab size L from Equation D-17, and then combining the factors with Equations D-15 and D-16. The procedures are described in the following paragraphs.

MODIFYING FACTOR FOR CONCRETE MODULUS E

Equations D-15 and D-16 were derived under the condition that the concrete modulus E = 4,000,000 psi. For other concrete modulus, the modifying factor can be established using either Equations D-6, D-9, or D-10. Equation D-6 was used because of its simplicity and accuracy. For E = 4,000,000 psi, the predicted subgrade k value is

$$k(E = 4{,}000{,}000\ \text{psi}) = \frac{13{,}342{,}400}{h^{4.0} \times w_{4{,}000{,}000}^{\exp(2.3A)} \times (4{,}000{,}000)^{\exp(2.3B)}}$$

and for other concrete modulus E

$$k(E) = \frac{13{,}342{,}400}{h^{4.0} \times w_E^{\exp(2.3A)} \times E^{\exp(2.3B)}}$$

For a given subgrade, the predicted subgrade k-value should be independent of the E modulus of the overlaying concrete slab. Consequently, the above two

expressions can be equated and the deflection w associated with the condition $E = 4,000,000$ psi can be written in terms of the deflection w associated with other E values:

$$\frac{13,342,400}{h^{4.0} \times w_E^{\exp(2.3A)} E^{\exp(2.3B)}} = \frac{13,342,400}{h^{4.0} \times w_{4,000,000}^{\exp(2.3A)} \times (4,000,000)^{\exp(2.3B)}}$$

This relationship results in the following equation.

$$w_{4,000,000} = w_E \left(\frac{E}{4,000,000}\right)^{e^{2.3(B-A)}} \qquad \text{(D-18)}$$

where

$w_{4,000,000}$ = the deflection associated with $E = 4,000,000$ psi ; should be used in lieu of the w in Equations D-15 and D-16.

w_E = the deflection associated with a given E value

It should be noted that Equation D-18 is derived based on conditions that the slab size L is 15 ft and the load P is 14,000 lb. The equation is used to modify deflection w in Equations D-15 and D-16 for situations when the concrete modulus E may be different from 4,000,000 psi, as both Equations D-15 and D-16 were formulated based on the condition $E =$ 4,000,000 psi . Data shown in Table D-4 (temperature differential equals zero) were again used to check the accuracy of Equation D-18. The computed k-values are presented in Table D-5. It is seen that Equation D-15 combined with Equation D-18 can appropriately account for the effect of the variation of concrete modulus E on predicted subgrade k-values.

MODIFYING FACTOR FOR SLAB SIZE L

Equations D-15 and D-16 were derived under the condition that the slab size L equalled 15 ft. For other slab sizes, the modifying factor can be established using Equation D-17. For $L = 15$ ft, the predicted subgrade k value is

Table D-5

Predicted Subgrade k-Values, Variable Concrete E Modulus

Deflection in.	Slab Thickness, h, in.	Concrete Modulus, E psi	Actual Subgrade Modulus, k, psi	Computed k	Percent Difference
0.04040	6	2,000,000	50	50.48	0.96
0.02735	6	2,000,000	100	105.37	5.36
0.01871	6	2,000,000	200	209.18	4.58
0.01140	6	2,000,000	500	512.45	2.49
0.03395	6	3,000,000	50	50.51	1.02
0.02288	6	3,000,000	100	106.30	6.29
0.01556	6	3,000,000	200	213.26	6.62
0.00946	6	3,000,000	500	524.67	4.93
0.03002	6	4,000,000	50	50.50	0.99
0.02020	6	4,000,000	100	106.57	6.57
0.01368	6	4,000,000	200	215.41	7.70
0.00829	6	4,000,000	500	533.21	6.64
0.02728	6	5,000,000	50	50.52	1.03
0.01835	6	5,000,000	100	106.67	6.66
0.01239	6	5,000,000	200	216.77	8.38
0.00748	6	5,000,000	500	540.33	8.06
0.02098	10	2,000,000	50	50.79	1.58
0.01410	10	2,000,000	100	98.23	-1.77
0.00949	10	2,000,000	200	190.72	-4.63
0.00566	10	2,000,000	500	468.34	-6.33
0.01776	10	3,000,000	50	51.18	2.35
0.01185	10	3,000,000	100	100.15	0.14
0.00797	10	3,000,000	200	194.79	-2.60
0.00473	10	3,000,000	500	482.99	-3.40
0.01590	10	4,000,000	50	50.80	1.59
0.01049	10	4,000,000	100	101.29	1.28
0.00705	10	4,000,000	200	197.32	-1.33
0.00418	10	4,000,000	500	490.39	-1.92
0.01467	10	5,000,000	50	50.06	0.11
0.00956	10	5,000,000	100	101.89	1.89
0.00641	10	5,000,000	200	199.32	-0.33
0.00379	10	5,000,000	500	498.04	-0.39
0.01330	15	2,000,000	50	54.64	9.28
0.00848	15	2,000,000	100	104.36	4.36
0.00563	15	2,000,000	200	196.37	-1.81
0.00334	15	2,000,000	500	467.65	-6.47
0.01188	15	3,000,000	50	51.95	3.90
0.00731	15	3,000,000	100	104.30	4.29
0.00476	15	3,000,000	200	202.42	1.20
0.00280	15	3,000,000	500	490.29	-1.94
0.01113	15	4,000,000	50	49.09	-1.81
0.00665	15	4,000,000	100	102.60	2.60
0.00424	15	4,000,000	200	205.71	2.85
0.00248	15	4,000,000	500	503.77	0.75
0.01066	15	5,000,000	50	46.50	-7.00
0.00623	15	5,000,000	100	100.09	0.09
0.00390	15	5,000,000	200	206.25	3.12
0.00225	15	5,000,000	500	517.35	3.46
0.01076	20	2,000,000	50	56.49	12.98
0.00633	20	2,000,000	100	106.53	6.52
0.00397	20	2,000,000	200	205.16	2.58
0.00231	20	2,000,000	500	484.64	-3.07
0.01009	20	3,000,000	50	52.20	4.39
0.00571	20	3,000,000	100	101.97	1.97
0.00345	20	3,000,000	200	206.29	3.14
0.00194	20	3,000,000	500	516.28	3.25
0.00973	20	4,000,000	50	48.89	-2.22
0.00538	20	4,000,000	100	97.19	-2.81
0.00316	20	4,000,000	200	203.47	1.73
0.00173	20	4,000,000	500	531.72	6.34
0.00952	20	5,000,000	50	46.16	-7.67
0.00518	20	5,000,000	100	92.69	-7.30
0.00298	20	5,000,000	200	198.49	-0.75
0.00159	20	5,000,000	500	539.95	7.99
Mean					1.70
R^2					4.36
SEE				13.28	

$$\log_e k(L = 15\text{ ft}) = 1.66 + 0.5234h - 0.0071 \times 15 + 6.398/15 + \log_e w_{15}$$
$$[-2.0636 + 0.506 \log_e h - 0.049(15) + 0.00079(15)^2$$
$$-0.00092\ (15)h] - \log_e(15)\ (0.1738h + 0.288 \log_e h)$$

and for other slab sizes L ,

$$\log_e k(L) = -1.66 + 0.5234h - 0.0071L + 6.398/L + \log_e w_L$$
$$[-2.0636 + 0.506 \log_e h - 0.049L + 0.00079(L)^2$$
$$-0.00092Lh] - \log_e L\ (0.1738h + 0.288 \log_e h)$$

For a given subgrade, the predicted subgrade k-value should be independent of the size L of the overlaying concrete slab. Consequently, the above two expressions can be equated and the deflection w associated with the condition $L = 15$ ft can be written in terms of the deflection w associated with other L values:

$$-0.0071 \times 15 + 6.398/15 + \log_e w_{15}\ [-2.0636 + 0.506 \log_e h$$
$$-0.049(15) + 0.00079 \cdot (15)^2 - 0.00092(15)h] - \log_e(15)(0.1738h$$
$$+0.288 \log_e h) = 0.0071L + 6.398/L + \log_e w_L\ [-2.0636$$
$$+0.506 \log_e h - 0.049L + 0.00079L^2 - 0.00092Lh]$$
$$-\log_e L\ (0.1738h + 0.288 \log_e h)$$

This relationship results in the following equation:

$$\log_e w_{15} = \frac{1}{C}\left[-0.0071\ (L - 15) + 6.398\ \left(\frac{1}{L} - \frac{1}{15}\right) - (\log_e L - \log_e 15)\right.$$
$$\left.(0.1738h + 0.288 \log_e h) + D \log_e w_L\right] \qquad \text{(D-19)}$$

where

$C = -2.621 + 0.506 \log_e h - 0.0138h$

$D = -2.0636 + 0.506 \log_e h - L(0.049 + 0.00092h) + 0.00079L^2$

w_{15} = the deflection associated with $L = 15$ ft ; should be used in lieu of the w in Equations D-15 and D-16

w_L = The deflection associated with a given L value

Equation 18 is used to modify deflection w in Equations D-15 and D-16 for situations when the concrete size L may be different from 15 ft, as both Equations D-15 and D-16 were formulated based on the condition $L = 15$ ft . It should be noted that Equation D-19 is derived based on the conditions $E = 4,000,000$ psi and $P = 14,000$ lb .

Table D-6 presents the subgrade k-values predicted by the combined Equations D-15 and D-19. Temperature differentials were assumed to be zero in Equation D-15. Although the predicted results are generally good, they are not as good as the results presented in Table D-5 in which the effect of concrete modulus E is considered.

MODIFYING FACTOR FOR BOTH CONCRETE MODULUS E AND SLAB SIZE L

Equations D-18 and D-19 can be combined into Equations D-15 and D-16 to account for the effects of concrete modulus E and slab size L . The combined equation has the form

$$w_{\left(\frac{15}{4,000,000}\right)} = \exp\left(\frac{1}{C}\left[-0.0071\ (L - 15) + 6.4\left(\frac{1}{L} - \frac{1}{15}\right) - (\log_e L - \log_e 15)\right.\right.$$

$$\left.\left.(0.1738h + 0.288\log_e h) + D\log_e\left[w_{LE}\left(\frac{E}{4,000,000}\right)^{\exp\left(\frac{B}{A}\right)}\right]\right]\right) \qquad \text{(D-20)}$$

where

$w_{\left(\frac{15}{4,000,000}\right)}$ = the deflection associated with $L = 15$ ft and $E = 4,000,000$ psi; should be used in lieu of the w in Equations D-15 and D-16

w_{LE} = the deflection associated with given values of slab size L and concrete modulus E

Computations were made to check the accuracy of Equation D-20. This was done by computing the deflections for various values of slab thickness h , concrete modulus E , slab size L , and subgrade modulus k ; temperature differential was assumed to be zero. The predicted k-values are presented in Table D-7. Compared with the results presented in Tables D-5 and D-6, Table D-7 reveals that not only the mean of the percent difference is larger, the standard deviation of the percent difference and the standard error of

Table D-6

Predicted Subgrade k-Values, Variable Slab Size L

Deflection in.	Slab Thickness, h, in.	Slab Size, L ft	Actual Subgrade Modulus, k, psi	Computed k	Percent Difference
0.03429	6	10	50	53.63	7.26
0.02241	6	10	100	110.24	10.23
0.01494	6	10	200	213.48	6.74
0.00874	6	10	500	512.55	2.51
0.03002	6	15	50	50.47	0.93
0.02020	6	15	100	106.51	6.50
0.01368	6	15	200	215.27	7.63
0.00829	6	15	500	532.82	6.56
0.02835	6	20	50	49.97	-0.05
0.01938	6	20	100	108.00	7.99
0.01339	6	20	200	221.26	10.62
0.00825	6	20	500	567.85	13.56
0.02764	6	25	50	46.86	-6.28
0.01918	6	25	100	102.42	2.41
0.01336	6	25	200	213.93	6.96
0.00825	6	25	500	572.33	14.46
0.02320	10	10	50	45.49	-9.02
0.01330	10	10	100	102.66	2.65
0.00815	10	10	200	211.74	5.86
0.00462	10	10	500	506.09	1.21
0.01590	10	15	50	50.76	1.52
0.01049	10	15	100	101.22	1.21
0.00705	10	15	200	197.17	-1.41
0.00418	10	15	500	[illegible]	-2.01
0.01456	10	20	50	48.33	-3.34
0.00983	10	20	100	98.80	-1.20
0.00665	10	20	200	202.71	1.35
0.00402	10	20	500	530.97	6.19
0.01391	10	25	50	44.79	-10.42
0.00943	10	25	100	95.31	-4.68
0.00646	10	25	200	199.78	-0.11
0.00398	10	25	500	535.79	7.15
0.02059	15	10	50	44.12	-11.76
0.01086	15	10	100	95.86	-4.15
0.00596	15	100	200	210.82	5.40
0.00296	15	10	500	574.20	14.84
0.01113	15	15	50	49.06	-1.87
0.00665	15	15	100	102.52	2.52
0.00424	15	15	200	205.52	2.76
0.00248	15	15	500	503.23	0.64
0.00879	15	20	50	51.95	3.90
0.00581	15	20	100	100.98	0.98
0.00392	15	20	200	199.25	-0.37
0.00233	15	20	500	527.59	5.51
0.00816	15	25	50	48.17	-3.65
0.00553	15	25	100	94.42	-5.58
0.00374	15	25	200	195.57	-2.21
0.00225	15	25	500	548.41	9.68
0.01994	20	10	50	52.19	4.38
0.01021	20	10	100	99.94	-0.05
0.00534	20	10	200	211.56	5.78
0.00241	20	10	500	610.58	22.11
0.00973	20	15	50	48.86	-2.27
0.00538	20	15	100	97.12	-2.88
0.00316	20	15	200	203.28	1.63
0.00173	20	15	500	531.10	6.22
0.00671	20	20	50	53.57	7.13
0.00413	20	20	100	103.25	3.25
0.00270	20	20	200	203.82	1.90
0.00160	20	20	500	529.21	5.84
0.00571	20	25	50	53.99	7.97
0.00379	20	25	100	99.10	-0.90
0.00257	20	25	200	195.21	-2.39
0.00153	20	25	500	552.50	10.50
Mean					2.78
R^2					6.14
SEE				25.68	

estimates are also larger, indicating that the residuals (the differences between the actual and the computed k-values) are scattered. This is not surprising because two modifying factors, i.e., slab size L and concrete modulus E, are incorporated in Equation D-20; the more the modifying factors are used, the more scattered are the residuals and the worse the predictions. It is also interesting to note in Table D-7 that for the case $L = 15$ ft the percent differences are relatively smaller than those for condition of different slab sizes. This is because the modifying factor in Equation D-19 becomes unity when $L = 15$ and thus modification is not involved and the predications are better. For a similar reason the percent differences are also smaller for the case where $E = 4{,}000{,}000$ psi.

It should be pointed out that the correlation equations developed in this Appendix are restricted by the following limitations:

a. The thickness of the concrete slab h ranges from 10 to 25 in.

b. The concrete modulus E ranges from 2,000,000 to 5,000,000 psi.

c. The slab size L ranges from 10 to 25 ft square.

d. The temperature differential ranges from -2°F to +3°F per inch of concrete.

e. The radius of the circular loaded area ranges from 6 to 24 in.

f. The subgrade modulus k ranges from 25 to 500 pci.

g. When the magnitude of the single-wheel load P is different from 14,000 lb, a multiplying factor $\left(\frac{P}{14{,}000}\right)^{\exp(2.3(0.29417 - 0.00733h))}$ should be used to correct the computed k-values (see Equation D-14).

Table D-7

Predicted Subgrade k-Values, Variable Concrete E Modulus and Variable Slab Size L

Deflection in.	Slab Thickness, h, in.	Concrete Modulus, E 10^6 psi	Slab Size, L ft	Actual Subgrade Modulus, k, psi	Computed k	Percent Difference
0.01630	10	2	10	100	114.40	14.39
0.00617	10	2	10	500	496.88	-0.62
0.01330	10	4	10	100	102.66	2.65
0.00462	10	4	10	500	506.09	1.21
0.01264	10	5	10	100	97.06	-2.93
0.00422	10	5	10	500	507.05	1.40
0.01291	10	2	25	100	88.82	-11.17
0.00554	10	2	25	500	479.76	-4.04
0.00943	10	4	25	100	95.31	-4.68
0.00398	10	4	25	500	535.79	7.15
0.00854	10	5	25	100	97.13	-2.86
0.00358	10	5	25	500	554.60	10.92
0.01068	20	2	10	100	122.85	22.84
0.00282	20	2	10	500	680.69	36.13
0.01021	20	4	10	100	99.94	-0.05
0.00241	20	4	10	500	610.58	22.11
0.01011	20	5	10	100	93.28	-6.72
0.00232	20	5	10	500	579.44	15.88
0.00514	20	2	25	100	88.58	-11.42
0.00208	20	2	25	500	473.68	-5.26
0.00379	20	4	25	100	99.10	-0.90
0.00153	20	4	25	500	552.50	10.50
0.00345	20	5	25	100	102.13	2.13
0.00139	20	5	25	500	577.25	15.44
0.01410	10	2	15	100	98.21	-1.78
0.00566	10	2	15	500	468.21	-6.35
0.01049	10	4	15	100	101.22	1.21
0.00418	10	4	15	500	489.94	-2.01
0.00956	10	5	15	100	101.80	1.80
0.00379	10	5	15	500	497.49	-0.50
0.00633	20	2	15	100	106.50	6.49
0.00231	20	2	15	500	484.38	-3.12
0.00538	20	4	15	100	97.12	-2.98
0.00173	20	4	15	500	531.10	6.22
0.00518	20	5	15	100	92.61	-7.38
0.00159	20	5	15	500	539.22	7.84
Mean						3.10
R^2						10.17
SEE					45.52	

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